HERIOT-WATT UNIVERSITY

A STUDY OF STAGGERED AND SHEAR-WALL TALL BUILDING SYSTEMS.

Thesis for Degree of PH.D.
in Structural Engineering
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To my parents for their dedication and encouragement during my entire education.

SYNOPSIS

Tall building structural systems, construction techniques and servicing requirements are discussed in an introductory chapter. The past research work relating to shear-wall systems and in particular to staggered-wall-beam systems is briefly reviewed and this present study summarized.

The research work presented here examines the behaviour of complex shear-wall, core and staggered-wall-beam systems.

The analysis techniques adopt the continuous connection method of analysis for continuous, uniform load-bearing elements, whilst simultaneously utilizing a plane-frame approach to analyse staggered-frame bents. It is often desirable if a non-uniform system can be substituted by a uniform system for analysis purposes. The possibility of replacing two adjacent staggered-frame-bents within a complete structure, by one coupled shear-wall unit at their mid-span, is examined as a possible method for a rapid assessment of structural behaviour in the early design stages of a staggered-wall-beam building.

An/

An experimental investigation evolved two techniques suitable for the fabrication of small-scale, complex high-rise reinforced concrete model structures. The methods were used to construct two single-bay staggered-wall-beam systems and two multi-bay structures ccmprising shear-walls, cores, wall-beams, columns and floor-slabs. The apparatus used to carry out static, dynamic and ultimate load tests on these models in addition to two perspex models, and the various procedures used to deflect and vibrate the models are described.

The results obtained from each series of tests are presented in graphical form and comparison is made with values obtained from the theoretical analyses.

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3.4/

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NOTATION

$A_1 A_2$	=	Cross-sectional area of walls 1 and 2
b	=	clear distance between walls
B, D	=	overall dimensions of coupled walls
E	=	Young's Modulus
G	=	Shear Modulus
h, H	=	storey and total height
I	=	moment of inertia of plane-wall
I_b	=	moment of inertia of connecting beams
I_1, I_2	=	moment of inertia of walls 1 and 2
I_w	=	moment of inertia of core wall
L	=	distance between centroidal axes of coupled walls
P_i	=	external load applied at level i
r	=	distance of shear centre from web of channel
t	=	wall thickness
T_i	=	twisting moment at level i
V	=	St. Venant torsion constant
W	=	end warping constant
x	=	height above base
y	=	horizontal deflection
J	=	torsional constant for plane-wall
d	=	breadth of a plane-wall
ν	=	Poisson's Ratio
ρ	=	mass per unit height
ω	=	circular frequency
λ	=	eigen-value

$[F]_k$ = matrix of flexibility coefficients of unit K

$[P]_k$ = matrix of lateral forces on unit K

$[P]$ = matrix of total lateral forces

$[S]_k$ = matrix of stiffness coefficients for plane-frame unit K

$[SL]_k$ = reduced matrix of flexibility coefficients for plane-frame unit K

$[T]_k$ = matrix of twisting moments on unit K

$[T]$ = matrix of total twisting moments

$[y]$ = matrix of deflection of reference axis

$[\theta]$ = matrix of structural rotations

$\{C\}$ = eigen-vector

CHAPTER 1

1.1 INTRODUCTION

The development of the high-rise construction industry has transpired partly because of prestige and egoistic purposes, but mainly because of the necessity to utilize available land area to its full potential. This trend in multi-storey buildings has helped meet some problems of urbanization such as the need for compactness of office accommodation and for residential housing in areas of high population density.

The concept of multi-storey structures is by no means a recent one. The Tower of Babel for example was built in 1563 B.C. This tower was solidly built and there was little risk of structural instability or progressive collapse.

As engineers became more knowledgeable in structural and material behaviour and characteristics so their design and building techniques became more sophisticated. This, combined with the necessity to reduce construction costs, and the reasons mentioned above has led to the rapid increase in the usage of tall, slender buildings over the last century. The "sky-scraper", once regarded as an American phenomenon can now be seen in many towns and cities throughout the world.

1.2/

1.2 TALL BUILDING TYPES

As buildings increase in height it becomes increasingly important to ensure adequate lateral stiffness to resist loads which may arise due to wind or seismic forces. The advanced technology used in design methods has given rise to many new forms of tall buildings which make efficient use of building materials and provide the required stiffness. These include precast, pre-stressed and cast-in-place reinforced concrete systems, structural steel frames, composite steel and concrete structures and masonry systems.

Earlier systems utilizing frames give good stability at an economical price for moderate heights. For tall buildings, however, these frames require large dimensions for columns and beams, are expensive in the use of reinforcing steel and limit the extent of column free space required in office buildings. Slab and wall systems are economical for high-rise structures up to about 30-storeys and ideal for residential purposes where numerous partitions are required but are less useful for commercial purposes which require floor plan flexibility and better internal communications. Also for greater heights the weight of closely placed walls reduces the viability of such a system.

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To make better use of space and materials, structural partitions which resist vertical and lateral forces were developed, i.e. "shear-walls". Box systems employing the high in-plane stiffness of shear-walls, combined with floor-slabs transferring lateral loads to vertical load bearing units are among the most common and least expensive methods of resisting wind-loads in high-rise construction. These walls serve the non-structural functions of providing fire-resistance, acoustic insulation and space-dividers. The regularity of such systems also permits extensive use of industrialised building techniques using either in-situ or pre-cast construction.

Central core systems containing elevators, stairwells and vertical ducts have been used to support suspended and cantilevered floors. The former system has been restricted and modified since the notable Ronan Point collapse. Some codes now stipulate that individual floors must have a self-supporting structure independent of that used for the floors above and below.

In the latter system each floor cantilevers from the core and like the suspended system allows a flexible architectural expression. This central core concept is particularly suitable for areas in which the foundations are in close proximity to the foundations of other buildings. The original concept of utilizing/

utilizing the service-core as the tube has developed into systems in which the tube is created by the outside wall, this has produced several structural forms such as the framed-tube, tube-in-tube, perforated-tube and bundled-tubes.

In the framed-tube building the exterior columns are very closely spaced and joined with rigidly connected spandrel girders resulting in a rigid exterior wall to provide lateral stability. The facade columns in the tube-in-tube structure form a peripheral framed tube around a central core, both interacting to resist the lateral forces. Sometimes it is considered undesirable to have exterior columns closely spaced. A structural system used on several occasions to avoid this connects the core shear walls to fewer exterior columns at one or more locations in the building such as mechanical floors or at roof level. In addition to increasing the lateral strength and stiffness of the building the rigid connection of the core to exterior columns also reduces problems associated with differential temperature expansion and contraction of the exterior exposed columns and the internal, temperature controlled core, as well as differential shortening. Several structural forms which have had wide application in steel are the terraced-infilled-frames, braced-frames and more recently the staggered-truss-system.

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The braced-frame usually consists of a beam and column framework infilled with X-bracing, K-bracing or the Warren Truss. The K-system and Warren Truss bracing have the architectural advantage of permitting doors and other openings at mid-span. Such frames may be used internally, in and around cores, or in the exterior facade. The difficulty of making the connections in reinforced concrete and the advantages of using shear-walls have generally eliminated the braced frame from concrete buildings. Staggered-Truss-Systems, which lend themselves to mass production, consist of a series of storey-high trusses placed such that at each bay the floor of the structure alternately rests on the top chord of one truss and the bottom chord of adjacent trusses. In addition to carrying the vertical load the trusses carry wind loads through the web-members and floor-slabs to the base, thus reducing the wind-bracing requirement. This system has been adapted to utilize reinforced-concrete wall-beams instead of trusses and referred to as the staggered wall-beam system.

The structural design process of tall buildings is developed in response to a wide range of imposed conditions and restraints. The final solution must be practical, utilitarian, aesthetically acceptable and in many/

many cases will result in a building comprising several of these structural forms mentioned above interacting with each other to resist all the imposed loads.

1.2.1 CONSTRUCTION TECHNIQUES

Construction techniques in both steel and concrete structures have developed simultaneously with building structural systems. Improved concrete technology and the development of strong light-weight aggregates have been fundamental to the growth in the use of concrete in tall buildings, whilst the variety of available steel sections and qualities, combined with improved shop and field welding techniques has allowed a greater flexibility in steel fabrication.

Repetitive structural elements which maintain uniform dimensions throughout a building has created a demand for more sophisticated formwork using larger, re-usable assemblies. Ganged forms which often combine column, beam, slab and wall elements in the same large piece of formwork and can be carried forward or up a building in large units, can improve efficiency in handling and placing in repetitive cast-in-situ concrete frames. A widely used form of shuttering, slip-forming, is ideally suited to core and shear-wall structures consisting of vertical elements continuous throughout most of the height of the building. Openings can be accommodated/

accommodated by careful placing of polystyrene moulds within the shuttering and future connections to beams and slabs allowed for in the reinforcement details. Prefabricated and prestressed elements are widely used to simplify and reduce shuttering and are available in a variety of sections; rectangular, T or double-T shaped joists, flat or ribbed slab-panels, spandrel beams, facade panels, flights of stairs and many other structural and non-structural units. Tolerances on such units are better than cast-in-situ parts and result in higher design stresses and in some cases lighter-weight members. The forces inherent in tall buildings necessitates strong joints and special care must be given to the continuity between successive units.

The most common method of erecting a steel framed structure is the tier by tier method in which each tier represents a column height of two or three building floors. Some medium height structures have been erected using a "push-up" construction. In this process the top floor is erected first at about ground level, jacked upwards, and additional steel placed underneath and attached to that previously jacked. Numerous types of hoisting equipment such as guy derrick, stiff-leg derrick, tower derrick and crawler cranes are utilized/

utilized in the lifting of structural steelwork.

Hybrid frame construction offers an unlimited combination of concrete and steel frames. The most prevalent of these are tall buildings having concrete cores and/or shear-walls and steel framing. The cores may be built completely to the top, using slip-forming, before starting steel erection or built simultaneously with the steel frame, one or the other lagging by a few floors.

Allowances must be made for construction tolerances allowed in both steel and concrete when designing connections. In many cases weld plates cast in the concrete with field welded connection angles has proved satisfactory. Top down construction involving fabrication of the entire structure at ground level and raising the building as construction proceeds has been used and adapted to several hybrid systems such as steel cores with concrete floor and precast concrete cores and steel floors.

The development of new structural systems such as the staggered-truss/wall beam has given more encouragement to the use of mass produced units, which have been largely restricted in the past to floor-systems.

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The importance of foundations in the construction of tall buildings cannot be overemphasized. Their height, with consequent heavy column loads, normally urban location with adjacent buildings, subways and pipe-networks require special consideration. Developments such as tie-backs, auger and drilled-in-caissons and bentonite slurry walls reflect the advance that has been made in foundation construction techniques.

1.2.2 MECHANICAL SERVICES

Mechanical services required in tall buildings include heating, ventilating and air-conditioning, water, gas and electrical supply and distribution, vertical transportation, sewerage and draining systems and fire-protection. Of these, vertical transportation is of prime importance, without elevator transportation, high-rise buildings would be impractical and uneconomical. The structural concept, floor-plan layout and architecture in general are strongly influenced by the elevator core systems and their inherent space requirements. As the height of a building increases the conventional elevator rapidly becomes inefficient due to the limited elevator speeds and/

and increasing core area. This necessitates the use of some other cost-effective transport system. A double-deck elevator system has been used effectively but has the disadvantage that all floor to floor heights must be exactly the same in order to permit accurate floor levelling. Any building space gained is offset by the extra cost of the system. Its efficiency decreases more rapidly than the conventional system as height increases and it is therefore not justified above fifty to sixty floors. Above this range the sky-lobby system is used. It involves express elevators up to various zones, with local elevators running just within these vertical zones so that local elevator bands are stacked one over the other. They can be either single or double-deck, have the advantage of reducing the number of elevators that terminate in the lower levels, subsequently increasing the rentable space and giving more latitude to the open planning of ground floors.

Technology offers a wide choice of heating, ventilation and air-conditioning systems, energy sources and distribution schemes, suitable for tall buildings. The optimum combination for any particular building is dependent on the potential use of the building, for example; apartments, offices, manufacturing facilities/

facilities, laboratories, hospitals or a combination of several of these functions. The general design of environmental systems such as those mentioned, although more complex, is not vastly different from that of other buildings. There are several features which require special consideration due to the height of the structure and they are; the hydrostatic pressure on piping systems and equipment, the stack effect in supply and exhaust air flows, the weight and space requirements of vertical risers and the location of the mechanical plant. None of these problems have proved insurmountable but may become more significant with future generations of taller buildings. Fire-Safety is one of the major criteria for the planning design and management of high rise structures. The greater number of occupants, the height of the building and the transmission of smoke and toxic gases through service shafts present greater risks in comparison with the more traditional buildings.

Shear walls, combined with horizontal and vertical zoning in a building can present an effective fire-barrier and minimize fire-spread. No additional protection such as asbestos fibre coating is required on shear-walls because of their inherent fire-resistance.

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The advantages of fire-resistant structural units and efficient fire-fighting and evacuation systems have been reflected in several major fires in tall buildings throughout the world.

1.3 BEHAVIOUR DURING EARTHQUAKES

The history of earthquake resistant structures built over the last twenty years indicates that the prime concern of the earthquake engineer has been to provide safety against collapse. Two characteristics considered essential in the structural design are
1) the ability to sustain high deformations without appreciable loss of strength, and
2) the ability to dissipate as quickly as possible a high input of energy. These requirements resulted in the development of the ductile moment-resisting frame. Whilst these ductile frames avoid major structural distress, the internal damage to the non-structural components and finishes can be extensive. (i.e. The Supreme Justice Building, the Banco Central and the Social Security Building during the Managua earthquake in 1972, similarly with buildings in Caracas during the 1967 earthquake).

Non-structural elements can constitute as high as 80% of the overall cost of a building, therefore it is essential that the design philosophy for earthquake-resisting/

resisting structures should accommodate damage control in addition to stipulating safety against collapse.

The behaviour of shear-wall structures during earthquakes occurring over the last fifeteen years has been very encouraging. During the Yugoslavian earthquake in 1963 the Party Headquarters (a shear-wall frame building) suffered only minor damage, similarly with the Indian Hill Medical Centre during the San Fernando earthquake in 1971. This pattern of behaviour has been observed in several earthquake areas, where neighbouring frame structures have suffered either severe structural/non-structural damage or total collapse. These examples and more have demonstrated that the twin requirements of safety and damage control can often be better met by the strength and stiffness inherent in shear-walls, coupled with sufficient ductility.

There have been occasions where structures with "interrupted shear walls" have failed to prevent large lateral displacements and have resulted in considerable structural damage. (i.e. The Olive View Hospital in San Fernando). This can be avoided by ensuring that the stiffness of the upper floors is continued through lower floors to ground level. Sliding movements along horizontal construction joints during earthquakes have also/

also created problems in the past. This potential weakness in shear-walls can be eliminated by ensuring adequate vertical reinforcement across coarse textured joints or where possible by continuous casting.
Lack of ductility is often the cause of a shear failure in the deep coupling beams. This ductility can be considerably improved by placing the principal reinforcement diagonally instead of in the conventional arrangement, as shown by Paulay (66).
It is necessary with such design details to provide ample ties in the reinforcement cage to confine concrete within the steel case and prevent buckling of the longitudinal steel.

The local environment, soil conditions and the earthquake history of a site necessitate an individual analysis of any tall building in a seismically active area. The design normally rests on the premise that the gravity load carrying walls will be the last ones to be damaged and that the structure will have an overall ductility factor of at least four. Several studies concerning coupled shear-walls have indicated that in order to achieve this much higher ductilities are required by individual structural members (67, 68, 75).

1.4/

1.4 HUMAN RESPONSE TO WIND-INDUCED MOTION

An important criterion to be considered in the design of buildings is the comfort of the occupants. The trend towards higher, lighter-weight and more flexible buildings with higher fundamental period, and smaller damping results in an increase in the dynamic response and wind-induced motion. The increase in horizontal motion, the magnitude of which is affected by wind speed and direction, building height, aspect ratio, mass distribution and local environment, has motivated studies of the threshold and more significantly the tolerance levels of human perception (11, 33, 51).

Experimental evidence has shown that human perception to horizontal linear and rotating motion can be caused by stimulation of the central nervous system and visual contact with the environment.

Recently Irwin (39) reviewed and analyzed existing data on human perception and produced a graph which gives acceptable r.m.s. values of building acceleration for the peak of the worst wind-storm in a typical five-year recurrance period over a frequency range of 0.56 to 20 Hz.

Restrictions of building movements to acceptable human tolerances is also of interest to the owner, consultant and contractor, since such movements can result in lease terminations and create a bad reputation for a particular building.

At/

At present a number of buildings are being instrumented to measure response to wind (11) and this, combined with further developments and refinements in the field of human response to building movements should result in a more comfortable environment for tall building occupants.

CHAPTER 2

2.1 The rapid increase in the use of shear-walls and in particular coupled shear-walls, illuminated the necessity for a better understanding of their behaviour. This stimulant for research has produced an extensive and comprehensive bibliography covering the elastic behaviour of two-dimensional shear-wall structures. These proposed analyses include finite element (54), finite difference (72), equivalent frame (56, 57, 78), laminar analysis (38, 41), numerical (17), semi-graphical (13) and grid techniques (29).

The development of these techniques has made available to the design engineer methods of assessing the structural behaviour of particular cases such as walls of variable thickness (16), irregularly positioned shear-walls (45), multi-bay systems (23, 38) and more recently the load distribution throughout complete three-dimensional structures (8, 19, 41, 79, 80) in addition to the cases of uniform, continuous, plane and coupled shear-walls.

Dynamic analyses and in-elastic analyses of tall buildings are largely restricted to research papers (12, 61, 74, 83, 22, 25, 27, 30, 85) and as yet, are incomplete.

Building/

Building systems, such as the staggered-truss/wall-beam system, are relatively new and consequently their behaviour under various loading conditions is not well documented.

The research work reported herein is directly associated with this sytem and since the behaviour of shear-wall systems is already well reviewed a literature survey of research work published during the last ten years which is pertinent to staggered-wall-beams is given here.

2.2 STAGGERED-WALL-BEAM-SYSTEMS

The staggered-truss system was developed by an architectural and engineering team at M.I.T. in 1966 (87). The system, initially conceived for steel structures, comprises a series of storey high trusses extending the entire width of the building, located in a staggered pattern such that while trusses on each floor are two bays apart, the floors still span only a single bay. The floor rests on the top chord of one truss and hangs from the bottom chord of a truss one bay away on the floor above. The trusses on any one floor are therefore two bays apart.

This/

This sytem has since been extended to incorporate reinforced concrete units, and has been referred to as the "Staggered-Wall-Beam-Framing-System". (Fig. 2.1). The system has been used on several occasions for the construction of reinforced-concrete, multi-storey residential buildings such as the Port De Mer Apartment Project in 1972 in Montreal, Canada and the Continental Plaza Hotel addition (1974) in Chicago, U.S.A. The structural principles are identical to those employed in the steel framing system; but the trusses are replaced by wall-beams which span the width of the building to lines of columns along exterior faces. Openings must obviously be provided in the walls for a corridor.

These walls make the building very rigid in the short direction. The shear forces through any one storey are resisted by the wall-beams which also induce axial loads in the columns. The horizontal shears from the walls on each floor are transferred through the floor slabs acting as stiff diaphrams, to the wall-beams on adjacent column lines on the floors above and below. The result is that only axial resistance is necessary in the columns for the short direction of the building. The columns may therefore be positioned with their webs perpendicular to the walls leaving their strong axis/

axis available to resist wind forces in the long direction of the building. In some cases the columns may be supplemented by shallow spandrell beams at the column lines, or by making elevator shafts, stairwells or a combination of both, load bearing to create interior shear walls.

In any architectural plan, wall-beams will occur on every second floor in each column line. This therefore leads to two typical floor layouts, which alternate between floors. Since balconies are mere extensions of the wall-beams the system offers a wide range of interesting facades, it is also economically competitive with the more conventional forms of construction for multi-storey buildings (26). This system would appear to offer an alternative form of construction which achieves aesthetic integrity and allows for change of use and interior reorganisation without structural alterations. Although not widely used at present this system does have potential in high-rise apartments and commercial property with a significant saving in overall construction costs and foundation requirements.

2.3 PAST RESEARCH WORK

The papers published during the previous ten years which are pertinent to the development of Staggered-Truss/

Truss/(Wall-Beam) systems are reviewed in this section in chronological order.

The Architectural Record first published a paper introducing the staggered-truss-system as developed at M.I.T. (87). This paper outlines the structural system and the philosophy behind its development. Emphasis is made on the possible economic, construction and architectural advantages of the system and a comparison of the quantity of steel required for a typical structure is made between the more conventional rigid and braced-frame systems and the new staggered-truss concept. The flexibility of the system to accommodate elevators, corridors and open planning are illustrated by several possible truss layouts.

Further information on the truss-system was presented by Goody, M.E., et al. (28) but this report was unavailable.

A detailed study of reinforced concrete wall-beam frames was described by Fintel, Bernard and Derecho (26). To the writer's knowledge this was the first instance of the staggered-truss concept being adapted to reinforced concrete elements. The report consists of four sections; Part One dealing with the architectural, mechanical/

mechanical and structural implications of the wall-beam system; Part Two - the analysis and design of a single wall-beam subjected to vertical loading; Part Three - a design example and Part Four - the economics of the wall-beam system.

In Part 2 a simply supported wall-beam with a single web-opening and subjected to vertical loading is used as the fundamental case. This analysis is extended to handle other support conditions and beams with up to three web-openings. Charts are presented to simplify the evaluation of moments and forces at critical sections along the span of a wall-beam subjected to distributed and point loads.

The method of analysis and use of the charts is illustrated in a design example in Part 3.

Part 4 of the report presents a comparison of three different structural systems, the flat plate, shear-wall and wall-beam system, in terms of the increase in material quantities and cost with increasing building height. Structural designs were carried out for 16, 26 and 36 storey sample buildings for the basis of comparison and material quantities for each of the designs presented in graphical and tabular form.

In/

In a separate paper Fintel (24) describes two tests on half-scale wall-beams designed according to A.C.I. 318 - 63 specifications. The specimens included top and bottom slabs to simulate an actual structural configuration. A uniform vertical load and additional point loads representing the weight of corridor walls were applied. The results indicated that the analytical and design procedures used in dimensioning the wall-beams were adequate. It is suggested by Fintel that the lateral load problem be approached by lumping together identical frames and considering each of these composite frames separately under lateral loads proportional to their respective tributary areas. Adjustments in the lateral load distribution between the various composite frames would then be carried out to achieve the condition of equal lateral deflection at each floor level (the slabs being assumed rigid in their own planes).

In a discussion of Fintel's paper, Barnard (5) comments on possible difficulties which can be encountered in the system and also the future modification of the concept to suit 'point-block' or square plan buildings with a central core.

Derecho (21) developed a program to carry out an elastic analysis of a multi-storey, single-bay wall-beam frame which/

which forms part of a staggered-wall-beam structure. The analysis yields the member-end forces of such a frame under gravity and/or lateral loads lying in the plane of the frame. The program uses an iterative method of solving simultaneous equations of equilibrium, formulated in terms of the joint displacement components. In the analysis for lateral loads, a provision has been incorporated in the program to account approximately for interaction between adjacent frames in the structure. Two cases are considered -

1) assuming that all the horizontal shear is taken by the wall-beams;
2) the more general case where the columns are stiff enough relative to the wall-beams to take a significant proportion of the total horizontal shear.

The program user must distribute the applied load between adjacent frame bents in the ratio of the column to wall-beam stiffness of a single storey.

The first structure designed and built using the staggered-truss-system was the 1300 Wilson Avenue Apartment project in the City of St. Paul, Minnesota, U.S.A. An account of the development process at M.I.T. and the following construction of this building is given by Bakke, Kloiber and Nuhn (4).

A/

A series of three papers published by the Nippon Kokan and Tokenaka Construction Company, Japan, (47, 48, 49) describe in detail tests carried out on individual trusses. The trusses were subjected to vertical and, lateral alternating cyclic, loading tests to assess their elastic and elastic-plastic structural behaviour, ultimate strength and ductility. The test results are compared with analytical results from the theoretical analyses outlined in the papers.

Gupta and Goel presented a study of the elastic and inelastic dynamic response characteristics of a truss framing system when subjected to a severe earthquake motion (31). By studying the behaviour of a single truss under a horizontal shear force, a simplification of this complex problem is made and an equivalent truss developed. Using this equivalent truss a mathematical model of the structure is developed and used to compute the inelastic dynamic response to a prescribed base motion, assuming a bilinear hysteresis behaviour for the principal members in the structure. The results of the computed response of a 20-storey structure, designed using current American Design Code Procedures (90), and subjected to 1.5 times the ground acceleration recorded during the first seven seconds of the El Centro 1940 earthquake are presented.

Hanson/

Hanson and Berg present a design philosophy for both the elastic limit and ultimate limit state design of staggered-truss-framing systems (32). The philosophy is described in detail and illustrated with the design of a 40-storey building in another paper by Hanson, Goel and Berg (34). Gupta studied the dynamic response of five staggered-truss-frames designed by current U.B.C. seismic design provisions (90). The response of these 10, 20 and 40 storey frames subjected to the El Centro and 1.5 times the El Centro 1940 N-S accelerogram is compared. A 40-storey structure from each of these papers was selected by Hanson and Berg to compare and illustrate the differences between the inelastic dynamic response of the structures designed by the two different procedures. The results of these analyses are presented and the suitability of the system for use in earthquake prone areas is indicated.

The three-dimensional behaviour of wall-beam structures was represented by a plane frame model in a paper by Mee, Jordan and Ward (60). An equivalent uniform member representing a typical wall was developed to idealize the wall-beam structure as a skeletal framework. A more accurate representation was suggested by/

by Rutenburg (3). For illustration purposes their own analysis, using finite element techniques, was limited to symmetrical wall-beams, but this could be extended to unsymmetrical configurations. Results were presented for a nine-storey test structure and gave good agreement with the predicted deflections.

A more simplified frame analogy is reported by Currie and Smith (20). Assuming rigid floor slab connection between frame bents, and including Michael's modification to allow for shear strains and local deformations (62), the authors found excellent agreement between predicted deflections and the experimental results found by Mee, Jordan and Ward on their plexiglass model (60).

2.4 PRESENT RESEARCH WORK

It is evident from the literature survey that most of the research carried out to date has been concerned with the truss-system. The papers which have reported studies on the wall-beam system are, like the truss system, restricted to structures comprising solely of staggered units.

The present research is concerned with the behaviour of single-bay S.W.B. units and integrated systems of single and coupled shear-walls, cores, S.W.B. units and/

and floor slabs subjected to lateral loading.

Static tests were carried out on models of the systems and natural frequencies, mode shapes, and damping present in the structures were determined by dynamic tests. The results of these tests are supplemented by those computed using approximate theoretical analyses presented in Chapter 3.

STAGGERED-WALL-BEAM - SYSTEM

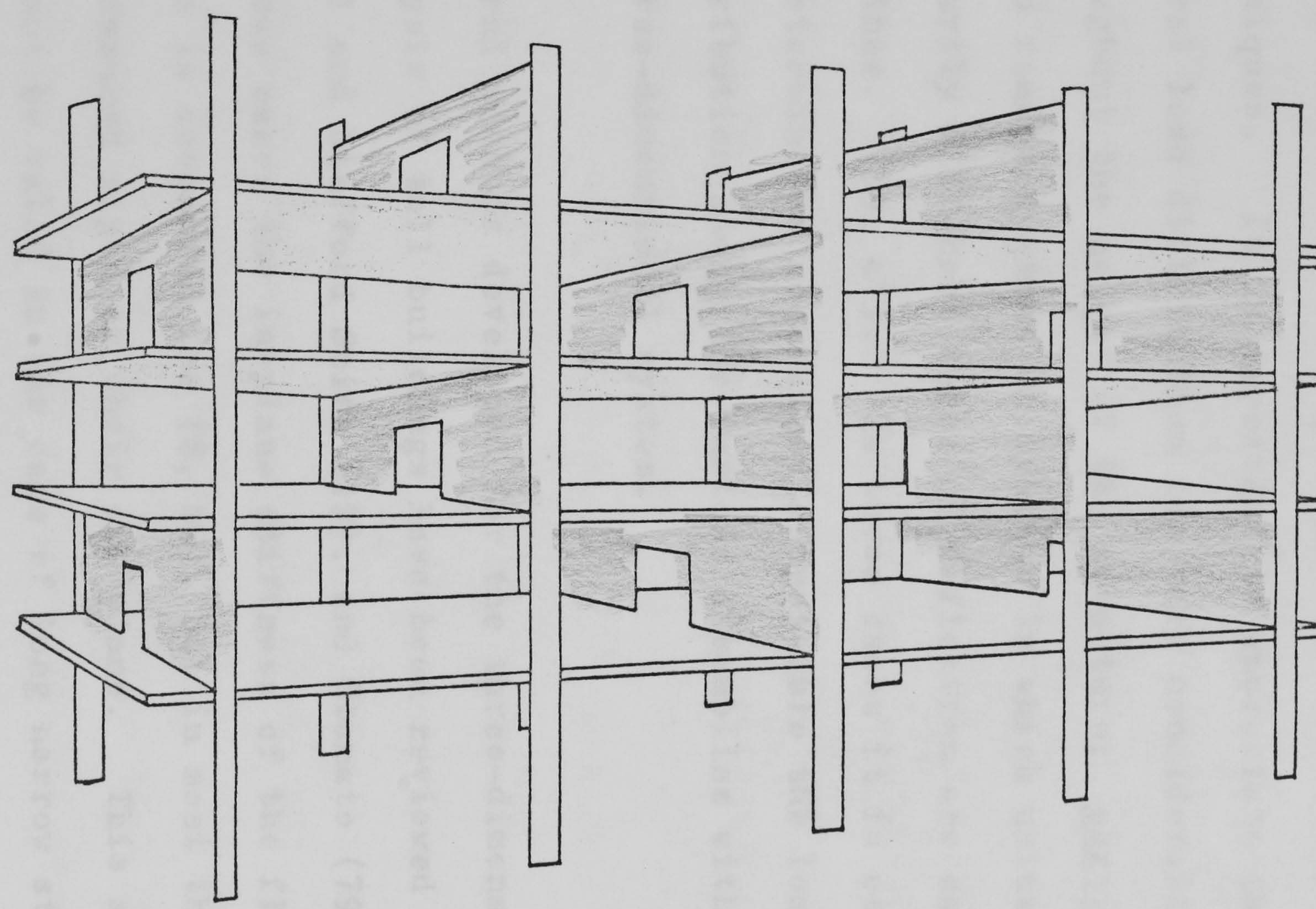


Fig. 2.1

CHAPTER 3

3.1 INTRODUCTION

Symmetrical tall structures can sometimes be simplified and analysed using standard two-dimensional, plane-frame, finite-element, grid and laminar analysis techniques. It is important to appreciate that the lateral load distribution can vary considerably throughout the height of the structure; particularly so in random systems or systems in which units of primarily different modes of deflection are coupled together. (19, 41). In these cases it is essential to determine as accurately as possible the load distribution among the various assemblies within a three-dimensional system.

Several methods developed for the three-dimensional analysis of tall buildings have been reviewed by Coull and Stafford Smith (19), and Stamato (79). In some cases the in-plane stiffness of the floor-slabs is accounted for (8, 40), but in most the slabs are assumed rigid in their own plane. This assumption may not be valid in the case of long narrow structures in which the load distribution can be influenced by the in-plane deformation of the slabs.

Attempts/

Attempts to obtain approximate solutions to the lateral load analysis of three-dimensional shear-wall/S.W.B. building systems are described in this Chapter. It is intended to assess their applicability to both preliminary and final design calculations. The methods of analysis used are standard plane-frame (84) and continuous connection techniques (41). These were selected in preference to finite-element or grid techniques, which are seldom justified in a design office because of the time and expense involved. In addition, the plane-frame approach is only used with those elements for which the continuous connection method is unsuitable, thus reducing the amount of computation that would otherwise be necessary in a full frame idealization.

3.2 PLANE-FRAME ANALYSIS OF A SINGLE FRAME BENT

The main component in the analysis of a frame-bent is the line element developed by MacLeod (57), as shown in Fig. 3.1. This element has two rigid ends and a flexible section which can deform axially, in bending and in shear. The rigid ends take account of the finite width of the walls and the eccentricity between the centroidal axes of the columns and walls. Each/

Each rigid part has three degrees-of-freedom and the element can be incorporated in a standard plane-frame program. The frame idealization for each type of frame bent is shown in Figs. 3.2 and 3.3 with an allowance for the local end deformations of the lintel beams included (62). These frames can be easily analysized by the direct stiffness method to yield joint displacements, beam and column moments, shears and axial forces.

Generally a building system will contain a number of identical frame bents, which can be lumped together as a composite frame for analysis purposes. The interaction forces between adjacent frames has been accounted for approximately by Derecho (21) by distributing the applied load between adjacent frame bents in the ratio of the column to wall-beam stiffness of a single-storey. (See Appendix C)

This method of analysis was carried out for Models 2, 3 and 4 with the load distribution as mentioned above and also with each frame bent subjected to half of the total applied load. The second analysis was carried out since the method suggested by Derecho assumes equal deflections of each frame bent/

bent, which is not the case in a single-bay asymmetric structure. A comparison of the results obtained using both loading cases is given in Appendix B.

3.3 THREE-DIMENSIONAL ANALYSIS OF STAGGERED AND SHEAR-WALL BUILDING SYSTEMS

In order to assess the structural behaviour of a complete building subjected to lateral loading, it is necessary to assess the lateral stiffness of its principal load bearing components. Generally the out-of-plane bending of walls, torsion of individual columns or beams and deformation in the plane of the floors can be safely ignored thus reducing the complexity of the problem.

Equations can then be established in terms of the contributions from each unit, expressing the equilibrium of the floors and imposed loading system with respect to translations along a pair of arbitrary orthogonal horizontal axes and rotations about an arbitrary vertical axis. Solution of these equations determines the translations and rotations at each floor and hence the in-plane displacement/

displacement of each unit, from which moments, shears and stresses can then be evaluated. A detailed explanation of this method of analysis can be found in reference (41) and only a brief resume will be given here.

The total lateral loading on a building can be represented by a series of point loads applied at a number of selected, equally spaced reference levels. If the distance between any two levels is 'h' then the point load at any level equals the integral of the loads over a height $^h/2$ above and below that level. For the arbitrary chosen vertical axis the resultant lateral load at any level will have an eccentricity Z such that the structure is subjected to a force P_i and twisting moment P_iZ at level 'i'. (See Fig. 3.4).

The total sum of the contributions from each load-bearing unit (k) at level 'i' must equal the total applied load at level 'i' to satisfy equilibrium conditions.

i.e./

i.e.

$$P_i = \sum_{k=1}^{m} P_{ik} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

$$T_i = P_i Z = \sum_{k=2}^{m} P_{ik} Z_k + \sum_{k=1}^{m} T_{ik} \qquad \ldots\ldots\ldots \quad (2)$$

where P_{ik} and T_{ik} are the load and twisting moment resisted by unit 'k' at distance Z_k from the chosen vertical axis; and m is the number of load bearing units.

The load-deflection relationship for any unit can be expressed in the form

$$[Y]_k = [F]_k \cdot [P]_k \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where $[Y]_k$ and $[P]_k$ are column vectors of deflections Y_{ik} and applied loads P_{ik} at the chosen set of reference levels. The matrix $[F]_k$ is the square matrix of flexibility coefficients f_{ijk}. If rigid floor-plan movement is assumed, then the total displacement of any unit 'k' at a distance Z_k from the chosen vertical axis can be expressed in the form

$$[Y]_k = [y] + Z_k[\theta] = [F]_k[P]_k \quad \ldots\ldots\ldots \quad (4)$$

where/

where $[y]$ and $[\theta]$ are column vectors of lateral deflections and rotations of the arbitrarily chosen axis of rotation (see Fig. 3.5). Having determined the lateral deflection of each unit, the lateral load on any unit can be expressed in the form,

$$[P]_k = [F]_k^{-1} \left([y] + z_k [\theta] \right) \text{..........} (5)$$

Substituting equation (5) into the equilibrium equations (1) and (2) give

$$T = \sum_{k=1}^{m} [F]_k^{-1} \left([y] + z_k [\theta] \right) z_k + \sum_{k=1}^{m} [k]_k [\theta] \text{.....} (6)$$

$$P = \sum_{k=1}^{m} F_k^{-1} \left([y] + z_k [\theta] \right) \text{..............} (7)$$

where $[T]$ and $[P]$ are column vectors of the total applied twisting moment and load respectively at each level 'i' .

$[k]$ is a square matrix of rotational stiffness coefficients.

The solution of the two simultaneous equations (6) and (7) yields the structural displacements and rotations about the chosen axis, for any system of applied loading causing bending and torsion of the structure.

i.e./

i.e.

$$[y] = \left[- \delta_2\delta_1^{-1}\delta_2 + \delta_3 + \delta_4\right]^{-1}\left[-\delta_2\delta_1^{-1}[P] + [T]\right] \qquad \ldots\ldots\ldots\ldots\ldots (8)$$

$$[\theta] = \left[-(\delta_4 + \delta_3)\delta_2^{-1}\delta_1 + \delta_2\right]^{-1}\left[-(\delta_4 + \delta_3)\delta_2^{-1}[P] + [T]\right] \qquad \ldots\ldots\ldots\ldots\ldots (9)$$

where $\delta_1 = \sum_{k=1}^{m} [F]_k^{-1}$ $\qquad \delta_2 = \sum_{k=1}^{m} [F]_k^{-1} Z_k$

$\delta_3 = \sum_{k=1}^{m} [F]_k^{-1} Z_k^2$ $\qquad \delta_4 = \sum_{k=1}^{m} [k]_k$

The deflection and force vectors for each unit can be determined from equations (4) and (5) and the twisting moments at each reference level for any unit from

$$[T]_k = [k]_k [\theta]$$

3.3.1/

3.3.1 COUPLED SHEAR-WALL UNITS

The relationship governing the deflection at any level due to a lateral point load applied at any level to coupled shear walls (Fig. 3.6), can be found using the continuous connection technique in which the discrete beams joining the walls are replaced by an equivalent continuous media with the same stiffness as the beams. The media is then assumed cut at its point of contraflexure and a compatability equation for the media set up and solved to give the load deflection relationship for the coupled walls as:-

$$\underset{o<x<x_i}{Y} = P_i \Big((3x_i x^2 - x^3)(1 - \alpha^2 L/\mu^2)/6 + \alpha^2 L/\mu^4 \left[x + \{\text{Sinh}\mu(H - x) - \sinh\mu H + \sinh\mu(H - x_i)(1 - \text{Cosh}\mu x)\} / \mu\text{Cosh}\mu H\right] \Big) / _{E.I} \quad \dots\dots\dots \quad (11)$$

$$\underset{x>x_i}{Y} = P_i \Big((3x_i^2 x - x_i^3)(1 - \alpha^2 L/\mu^2)/6 + \alpha^2 L/\mu^4 \left[x_i + \{\text{Sinh}\mu(H - x) - \sinh\mu H + \sinh\mu(H - x_i) + \text{Cosh}\mu h\text{Sinh}\mu(x - x_i) - \sinh\mu(H - x)\text{Cosh}\mu X\} / \mu\text{Cosh}\mu H\right] \Big) / _{E.I} \quad \dots\dots \quad (12)$$

where the parameters α and μ are defined as:-

$$\alpha^2 = 12LI_b / \ b^3h(2I_1 + I_2)$$

$$\mu^2 = \alpha^2 \left[2L^2 + (2I_1 + I_2)/A_1L\right]$$

for/

for walls with two symmetrical bands of openings and

$$\alpha^2 = 12LI_b/b^3hI$$

$$\mu^2 = \alpha^2 \left[L + (A_1 + A_2)I/A_1A_2L\right]$$

for walls with a single band of openings.

The bending flexibility matrix for a coupled shear-wall unit can be formed using equations (11) and (12), and on inversion give the stiffness matrix. This matrix can now be used as described previously to assess the contribution of a coupled-shear wall unit to the overall stiffness of a complete structure, and to evaluate the unit moments, shears and stresses from the structural displacements. The effective widths of the floor slabs which act as beams coupling walls was estimated using the curves presented by Quadeer and Stafford Smith (71) and the local end deformations of the lintel beams taken account using the method suggested by Michael (62).

3.3.2 UNIFORM PLANE SHEAR-WALL UNITS

Uncoupled cantilever units have a corresponding load-deflection relationship from which bending flexibility and hence stiffness matrices can be formulated,

i.e.

$$Y_{o<x<x_i} = P_i(3x_i x^2 - x^3)/6EI \quad \dots\dots\dots\dots\dots\dots\dots (13)$$

$$Y_{x>x_i} = P_i(3x_i^{\,2}\, x - x_i^{\,3})/6EI \quad \dots\dots\dots\dots\dots\dots\dots (14)$$

The torsional stiffness of uniform plane units can be expressed in finite difference form as

$$T_{ik} = K_{ik} \frac{1}{2d} (\theta_{i+1} - \theta_{i-2}) \quad \dots\dots\dots\dots\dots\dots\dots (15)$$

where T_{ik} = twisting moment at level x_i

d = height interval between reference levels

K_{ik} = torsional stiffness of the element

For the topmost level x_n, an alternative expression must be used,

i.e.

$$T_{nk} = K_{nk} \frac{1}{d} (\theta_n - \theta_{n-1}) \quad \dots\dots\dots\dots\dots\dots\dots (16)$$

Equations (15) and (16) enable a matrix of torsional stiffness coefficients for a plane unit to be formulated.

3.3.3/

3.3.3 TORSIONAL STIFFNESS OF CORE UNITS

The equation for the rotation at any level of coupled channels (Fig. 3.7) due to a twisting moment applied at any level, also found using the continuous connection technique is:

$$\theta_{0<x<x_i} = -\phi T_i \Big(-x - \mathrm{Sinh}\,\alpha(x_i - x)/\alpha + \big[\mathrm{Sinh}\,\alpha H + \mathrm{Sinh}\,\alpha(H - x)(\mathrm{Cosh}\,\alpha x_i - 1) - \mathrm{Sinh}\,\alpha(H - x_i)\big]/\alpha\,\mathrm{Cosh}\,\alpha H \Big)/\alpha^2 \quad \ldots\ldots\ldots\ldots (17)$$

$$\theta_{x>x_i} = -\phi T_i \Big(-x_i + \big[\mathrm{Sinh}\,\alpha H + \mathrm{Sinh}\,\alpha(H - x)(\mathrm{Cosh}\,\mu x_i - 1) - \mathrm{Sinh}\,\alpha(H - x_i)\big]/\mu\,\mathrm{Cosh}\,\mu H \Big)/\alpha^2 \ldots\ldots\ldots\ldots (18)$$

where ϕ and α are defined as

$$\phi = \frac{2}{4W + EI_W(D + 2r)^2}$$

$$\alpha^2 = \frac{4V + 24EI_b DB^2(D + 2r)/hb^3}{4W + EI_W(D + 2r)^2}$$

$$V = \frac{Gt^3}{3}(B + D - b - t)$$

$$W = \frac{EtB^2}{192}\left(\frac{D-b-t}{2}\right)^3 \left(1 + \frac{tB^3}{4I_W}\right.$$

$$r = \frac{3\,(D - a - b)^2}{4\,\big[3(D - a - b) + B\big]}$$

Equation (17) and (18) enable the torsional stiffness for coupled channels to be formulated and their subsequent contribution to the overall torsional stiffness and the entire structure accounted for.

3.3.4

3.3.4 STAGGERED FRAME BENT UNITS

In any S.W.B. system there are two typical frame bents as shown in Figs. 3.2, 3.3. The technique used here for assessing the lateral stiffness of such frame bents enables a three-dimensional laminar/frame analysis to be carried out on combined shear-wall/S.W.B. systems. This method considerably reduces the computer time and effort that would otherwise be required in a full equivalent plane-frame, space-frame or finite element analysis.

The complete structure analysis described in section 3.3 deals with matrices of size (N x N), where N is the chosen number of reference levels. It is necessary, when using the analysis outlined here, that N equals the total number of levels since a frame bent is not uniform throughout the height and it is difficult to reduce to an equivalent frame with a smaller number of storeys. The procedure for formulating the (N x N) lateral flexibility matrix $SL[\diagdown]_k$ is as follows:-

1) Using a standard plane-frame program, the stiffness matrix $S[\diagdown]_k$ for a single frame-bent is developed (6N x 6N) employing the element mentioned in section 3.2. (three-degrees-of-freedom at each node and two nodes at each level).

2)/

2) A unit lateral load is applied at level 1 and the corresponding structural displacements evaluated.

3) From the frame displacements found in step 2, the lateral deflection at each level is selected to establish a vector of N elements.

4) The vector created in step 3 constitutes the first column of the lateral flexibility matrix $SL[\diagdown]_k$.

5) Steps 2, 3 and 4 are repeated for each level in turn with each vector representing the corresponding column in the matrix $SL[\diagdown]_k$.

The resulting matrix must obviously be symmetrical if the system is linear and this can be checked on completion of the matrix $SL[\diagdown]_k$. Inversion of this matrix yields the lateral stiffness of a single frame-bent in a matrix form which is consistent with the other load-bearing units within the structure; this can be used in the same manner to develop the matrices for the entire structure. The force vector for a frame-bent unit can be determined using this matrix after evaluating the structural displacements/

displacements and rotations. This force vector is subsequently used as data input to a plane-frame analysis to calculate the moments, shears and axial forces in the frame-bent members.

3.4 PRELIMINARY ASSESSMENT OF STRUCTURAL BEHAVIOUR

It is often convenient if a non-uniform structure can be replaced by a uniform system to rapidly obtain an approximate solution and give a preliminary assessment of structural behaviour. The continuous connection technique is ideally suited to this type of analysis since the problem can be made as lengthy or as brief as the designer chooses.

The obvious disadvantage in the analysis given above (Section 3.3) is that it is difficult to reduce the chosen number of reference levels. It was decided to investigate the possibility of replacing two adjacent frame bents by one coupled shear-wall unit placed mid-way between them as in Figs. 3.8 and 3.9, for analysis purposes. This would enable a rapid indication of the probable structural deflections and rotations (using the method outlined in Section 3.3) and hence assist in any modification of member sizes before carrying out a detailed analysis.

The/

The dimensions used for the coupled shear-wall unit are those of a wall-beam at any level, assuming wall-beams to be identical throughout the structure. A standard three-dimensional analysis, as described in Chapter 3, is then carried out using only shear-wall units to yield deflections and rotations.

3.5 COMPUTATION

All computation was carried out on an I.B.M. 370 computer facility offered by Edinburgh University, Department of Computer Science. Programs were read into the computer on punch cards and output was produced on a line-printer. If required, a graph plotter, magnetic tapes, and storage disc facilities were available.

Since the writer had previously been introduced to a suitable engineering computer language, i.e. FORTRAN, this was used in compiling all programs.

The structure of the programs were such that subroutines existed to evaluate bending stiffness matrices, load and displacement vectors, moments, shears/

shears and stresses, for plane and coupled shear-walls, and staggered frame bents. Similar subroutines were compiled to evaluate the torsional stiffness matrices of plain-walls and coupled channel units. Individual subroutines to carry out inversion, addition, subtraction and multiplication of matrices were also available.

The main program for the three-dimensional analysis of Models No. 5 and 6 incorporated four basic sections:-

1) Formulation of the structural bending stiffness matrix

2) Formulation of the structural torsional stiffness matrix

3) Evaluation of structural deflections and rotations

4) Evaluation of element deflections, forces and stresses.

Checks on the equilibrium of the applied forces and calculated element forces were carried out after step 4. The structure of the program enabled modifications/

modifications without any major reconstruction and each subroutine could be 'run' seperately and checked by hand for a simple case. Advantage could also be taken of identical units within the structure by lumping them together and evaluating the element stiffness matrices for these types of elements once only.

3.6 DYNAMIC ANALYSIS

Ref.(12) presents a simple approximate analysis for the free bending and torsional vibrations of regular symmetrical shear-wall building comprising a number of coupled shear-wall assemblies and service cores.

The analysis employs the continuous connection method to formulate the differential equation for the deflection of a statically loaded coupled shear-wall cantilever system and subsequently d'Alembert's principle to form the governing equation which describes the dynamic behaviour of that system. Further manipulation of this governing equation results in a matrix characteristic equation of the form $[A]\ \{C\} = \lambda[B]\ \{C\}$ which can be solved for λ and $\{C\}$ using standard eigen-value techniques. Determination/

Determination of the eigen-value (λ) and eigen-vectors ($\{C\}$) lead to estimates of the first few natural frequencies and corresponding mode-shapes.

The analysis, which is suitable for hand calculation, is described in detail in Ref.(12) and is therefore only briefly outlined below.

A plane system consisting of a pair of coupled shear-walls which are constrained by floor-slabs to act in conjunction with a single cantilevered wall is shown in Fig. 3.10. The action of the floor slabs is simulated by a series of pin-ended links, which transmit axial forces only. Using the continuous connection method, the discrete set of connecting beams and pin-ended links can be replaced by continuous media of flexural rigidity EI_c/h per unit height for the beams and axial force of intensity n_2 per unit height for the pin-ended links as in Fig. 3.11.

The differential equation for the deflection of the statically loaded coupled shear-wall cantilever system can then be set up employing the moment curvature relationship:-

$$E(I_1 + I_2 + I_3)\,\frac{d^2y}{dx^2} = M - lN \quad \ldots\ldots\ldots\ldots\ldots \text{(a)}$$

the structural parameters and imposed forces,

$$EI\,\frac{d^4y}{dx^4} - EI\alpha^1\,\frac{d^2y}{dx^2} = \frac{d^2M}{dx^2} - \beta^1 M \quad \ldots\ldots\ldots\ldots \text{(b)}$$

where M is the static moment at any height 'x' due to the imposed forces, and N is the axial force in each coupled wall.

The dynamic equation of this sytem may be derived from this expression provided that the external forces are augmented by the inertia forces. In the case of free vibrations, as studied here, the external forces are zero and hence only inertia forces need be considered, the lateral force intensity is thus given by

$$\frac{\partial^2 M}{\partial^2 x^2} = w(x) = -\rho\,\frac{\partial^2 y}{\partial t^2} \quad \ldots\ldots\ldots\ldots\ldots \text{(c)}$$

On changing to partial derivations to take account of the two variables x and t, rearranging and differentiating equation (b) twice with respect to x, and substituting for $\frac{\partial^2 M}{\partial x^2}$ from (c) the governing equation is found to be

$$\frac{\partial^6 y}{\partial x^6} - \alpha^1 \frac{\partial^4 y}{\partial x^4} = - \frac{\rho}{EI} \left(\frac{\partial^4 y}{\partial x^2 \partial t^2} - \beta \frac{\partial^2 y}{\partial t^2} \right)$$

where ρ = mass per unit height of the system

α^1, β^1 = variables dependent on structural dimensions.

Expressing the deflection y as a function of height (x) and time (t) such that

$$y = \phi(x) \eta(t)$$

enables two equations involving ordinary derivations to be obtained, i.e.

$$\frac{d^2\eta}{dt^2} + w^2\eta = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (d)$$

and

$$\frac{d^6\phi}{dx^6} - \alpha^1 \frac{d^4\phi}{dx^4} = w^2 \frac{\rho}{EI} \left(\frac{d^2\phi}{dx^2} - \beta^1 \phi \right) \quad \ldots\ldots\ldots (e)$$

where w = circular frequency.

A numerical solution of partial differential equations (the Galerkin Technique) is used to form the matrix characteristic equation $[A]\{C\} = \lambda [B]\{C\}$ from the frequency equation (c) and the two algorithms given from this, (the elements of matrices A and B), are evaluated.

i.e./

i.e.

$$a_{jk} = \frac{1}{2}\left\{\left(\frac{m\pi}{2}\right)^6 + \alpha\left(\frac{m\pi}{2}\right)^4\right\} \qquad j = k$$

$$= 0 \qquad j \neq k$$

and

$$b_{jk} = 1 + \beta\left\{1.5 + \frac{4}{m\pi}(-1)^{(m+1)/2}\right\} + \frac{1}{2}\ \frac{m\pi}{2}^{\ 2} \qquad j = k$$

$$= 1 + \beta\left\{1.0 + \frac{2}{\pi}\left[\frac{1}{m}(-1)^{(n+1)/2} + \frac{1}{n}(-1)^{(n+1)/2}\right]\right\} \qquad j \neq$$

where $m = 2j - 1$; $n = 2k - 1$

The matrix characteristic equation may therefore be set up and solved for λ and $\{C\}$ using standard techniques, and values of the natural frequencies and modes of vibration determined.

The previous equations may be applied directly to the analysis of three dimensional regular symmetric cross-wall structures consisting of cantilevered box cores and assemblies of identical coupled shear-walls. Model Number 6 and the simplified representation of Model Number 5 of this present study can be included in this category.

Two sets of hand calculations were carried out for each model to determine the first three natural frequencies and mode shapes; firstly for flexural vibrations and secondly for torsional vibrations.

Flexural/

Flexural Vibrations. If we assume rigid floor-plan movement, then for no rotation of the structure, all wall assemblies undergo the same horizontal deflection at any level. The three-dimensional structure may then be replaced by an equivalent plane structure, obtained by assembling the parallel wall elements in series with rigid pin-ended links at each floor level to simulate the floor slabs. The spatial structure is replaced by the simpler equivalent plane system of Fig. 3.10, in which the stiffness of the single pair of coupled walls is equal to the sum of the stiffnesses of all coupled wall assemblies, and the flexural rigidity of the individual cantilever element equals the sum of the cantilever elements in the structure.

The natural frequencies and modes of flexural vibration then follow directly from the previous analysis.

Torsional Vibrations. Under pure torsional oscillation, a symmetrical building will rotate about its central vertical axis of symmetry and the horizontal deflection of any wall will be proportional to/

to its distance from the central axis. For a linear system, the forces in identical coupled wall assemblies will also be proportional to their distances from the axis of symmetry.

In a similar manner to that for flexural vibrations the spatial structure may be transformed into an equivalent plane system as before. Care must be taken to ensure that deflections, forces and wall properties of all assemblies are transformed into a common datum plane.

As before stiffnesses and flexural rigidities of all coupled-wall and cantilever assemblies must be summed and the analysis carried out using the formulae previously given.

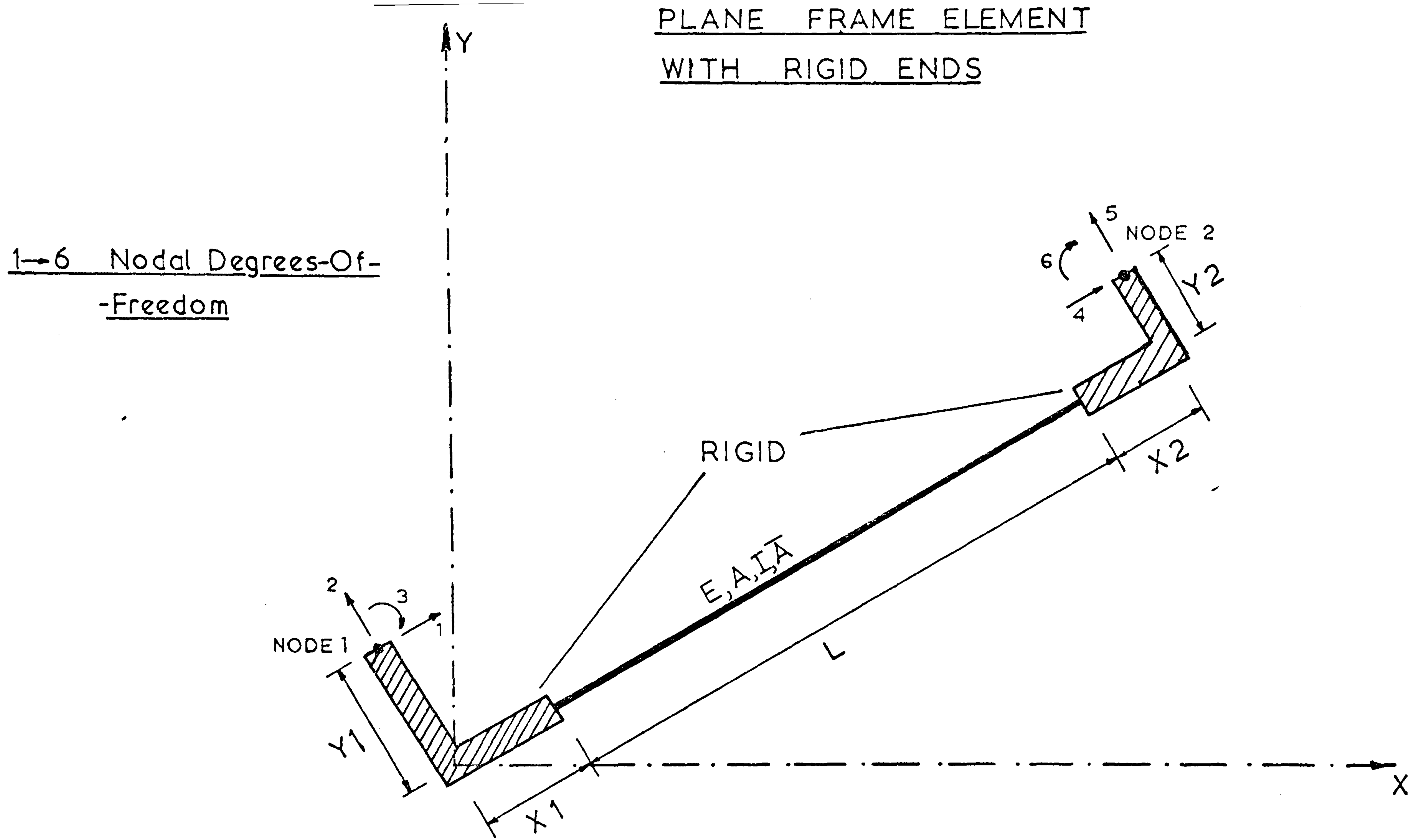

PLANE FRAME ELEMENT
WITH RIGID ENDS
1→6 Nodal Degrees-Of-
-Freedom
Y
X
NODE 1
NODE 2
1
2
3
4
5
6
RIGID
E,A,I,Ā
L
X1
X2
Y1
Y2

Fig. 3.1

$$
\begin{array}{c|cccccc}
 & 1 & 2 & 3 & 4 & 5 & 6 \\
\hline
1 & A/L & & & & & \\
2 & 0 & \frac{2\alpha_1}{L^2} & & \text{SYMMETRIC} & & \\
3 & -\frac{A.Y1}{L} & -\alpha_1.\alpha_3 & \frac{A.(Y1)^2}{L} + \alpha_1.\alpha_3.X1 + K_1 & & & \\
4 & -\frac{A}{L} & 0 & \frac{A.Y1}{L} & \frac{A}{L} & & \\
5 & 0 & -\frac{2\alpha_1}{L^2} & \alpha_1.\alpha_3 & 0 & \frac{2\alpha_1}{L^2} & \\
6 & \frac{A.Y2}{L} & -\alpha_1.\alpha_4 & \frac{A.Y1.Y2}{L} + \alpha_1.\alpha_2 + K_2 & -\frac{A.Y2}{L} & \alpha_1.\alpha_4 & \frac{A.(Y2)^2}{L} + \alpha_1.\alpha_4.X2 + K_2
\end{array}
$$

ELEMENT STIFFNESS MATRIX

$$K_1 = \frac{2I.(0.34+B)}{L.(2B+0.17)}$$

$$K_2 = \frac{I.(0.34-2B)}{L.(2B+0.17)}$$

$$B = \frac{2I.(1+\nu)}{\bar{A}.L^2}$$

$$\alpha_1 = K_1 + K_2$$

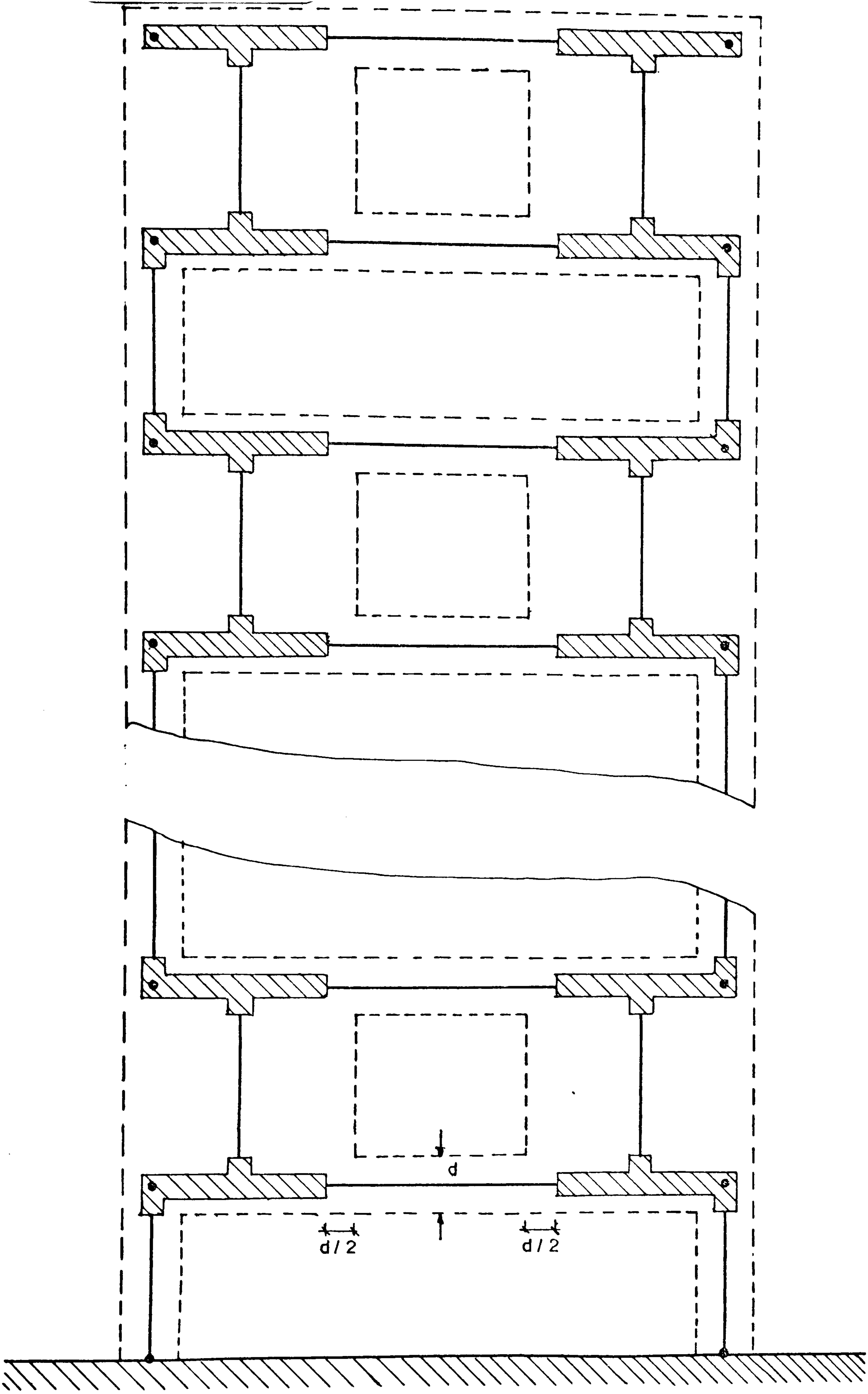
$$\alpha_2 = \frac{(2X1.X2)}{L^2} + \frac{X1}{L} + \frac{X2}{L}$$

$$\alpha_3 = \frac{2X1}{L^2} + \frac{1}{L}$$

$$\alpha_4 = \frac{2.X2}{L^2} + \frac{1}{L}$$

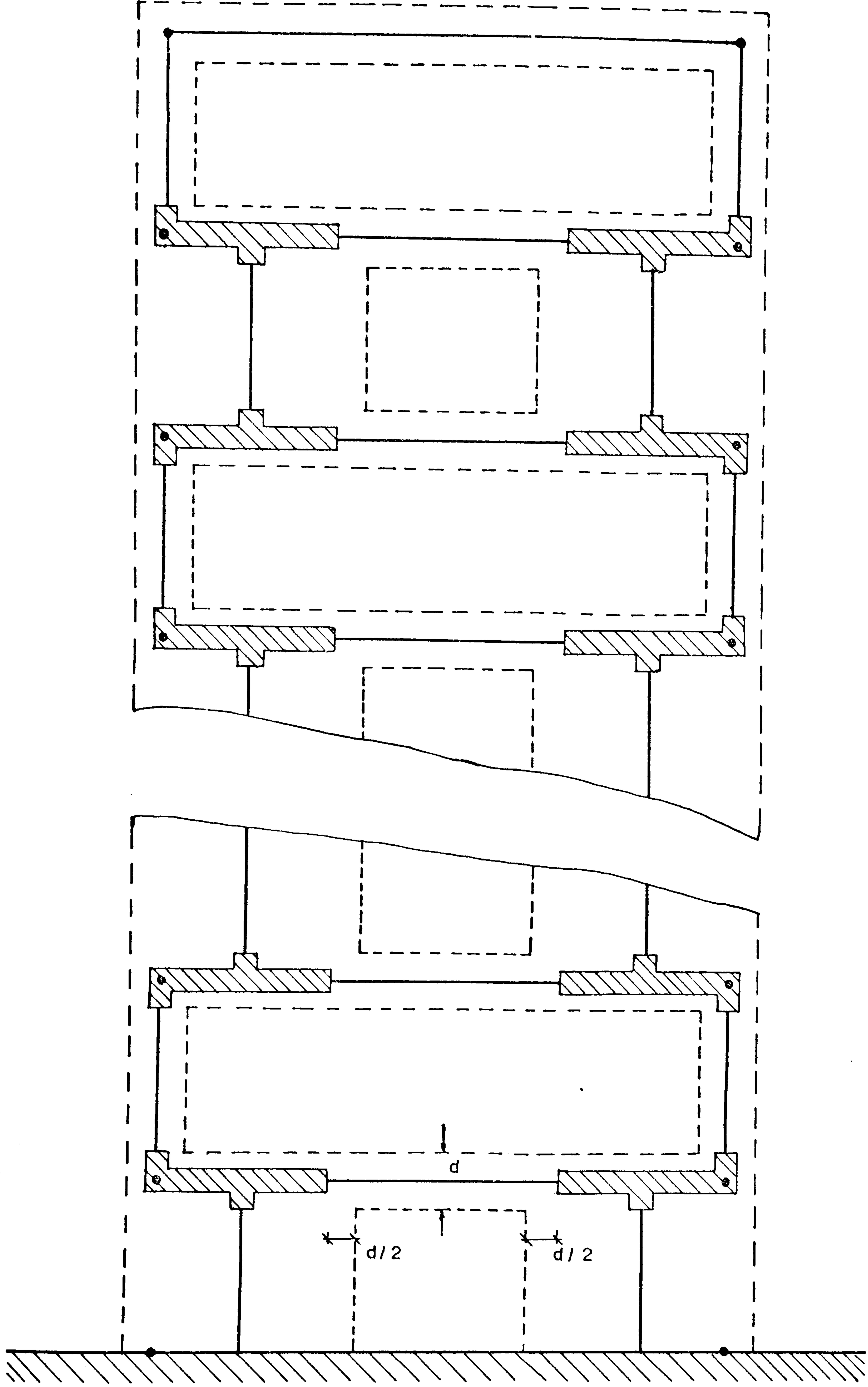
$\bar{A}$ = Equivalent shear area

ν = Poissons's Ratio



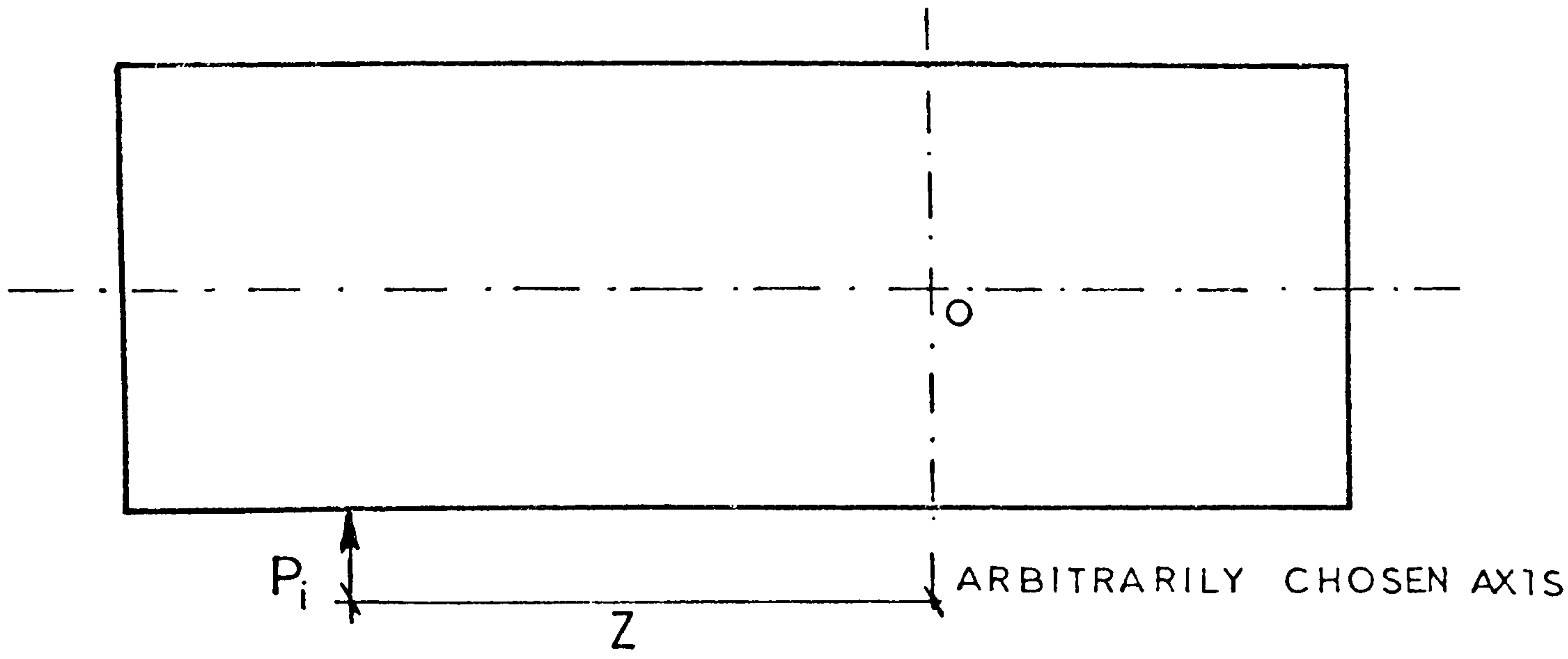
FRAME-BENT – TYPE A (Columns at base)

Fig. 3.2



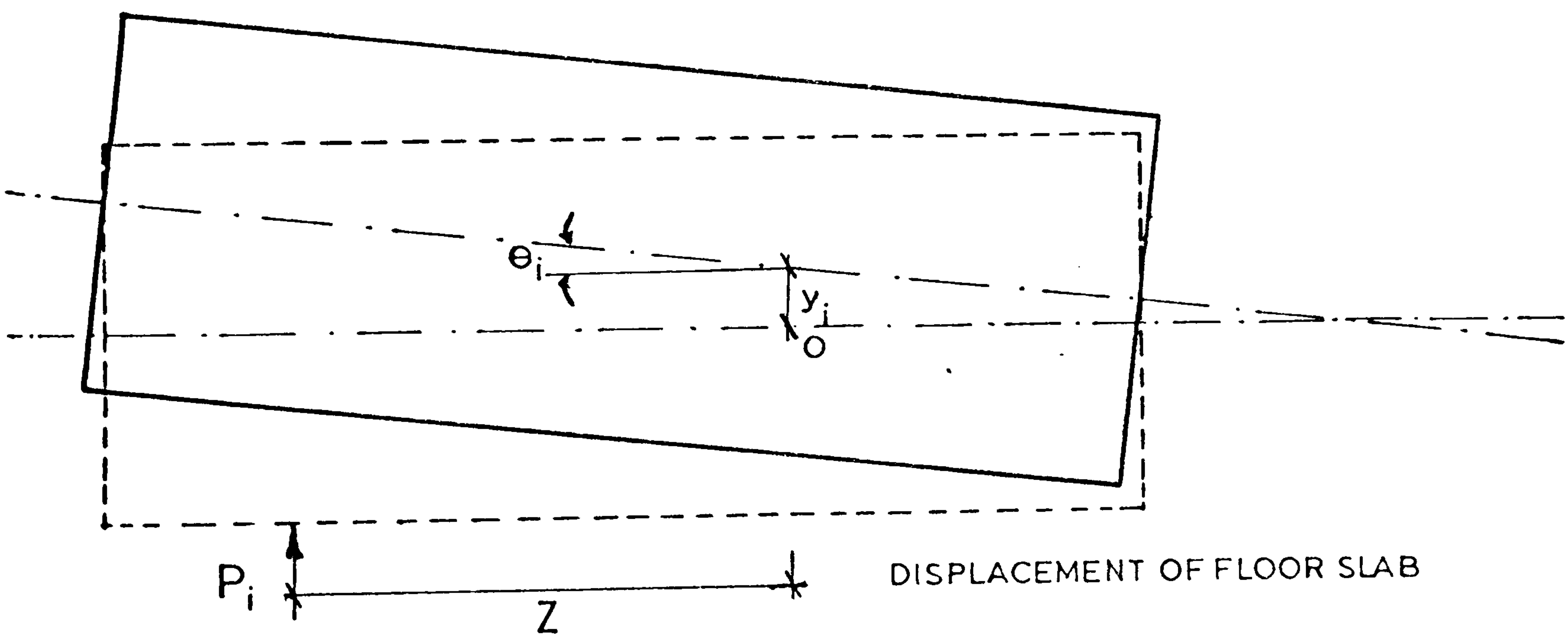
FRAME - BENT — TYPE B (Walls at base)

Fig. 3.3



o
P_i
ARBITRARILY CHOSEN AXIS
Z

Fig.3.4



θ_i
y_i
o
P_i
Z
DISPLACEMENT OF FLOOR SLAB

Fig.3.5

COUPLED SHEAR WALL UNIT

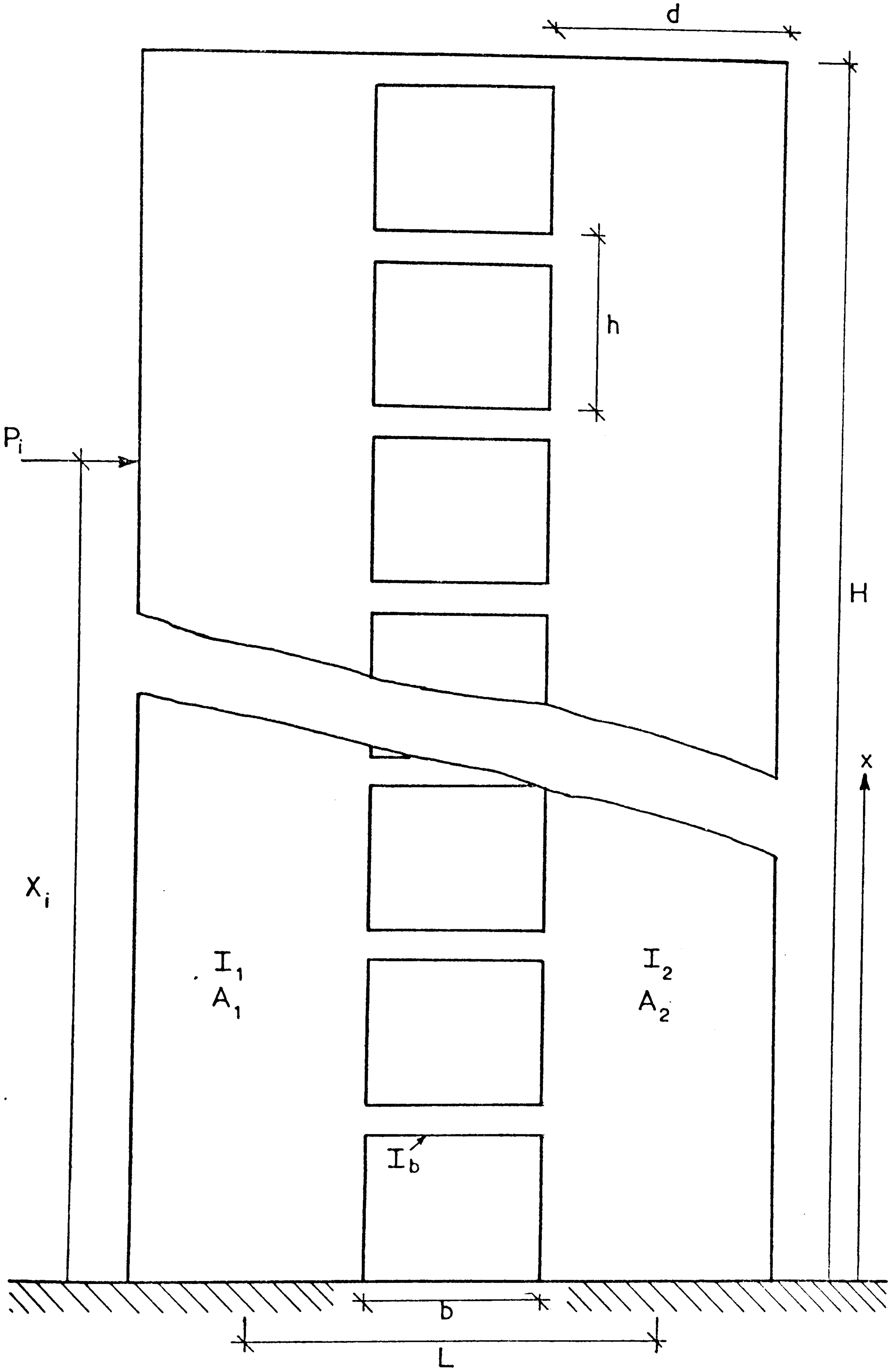


Fig. 3.6

COUPLED CHANNEL UNIT

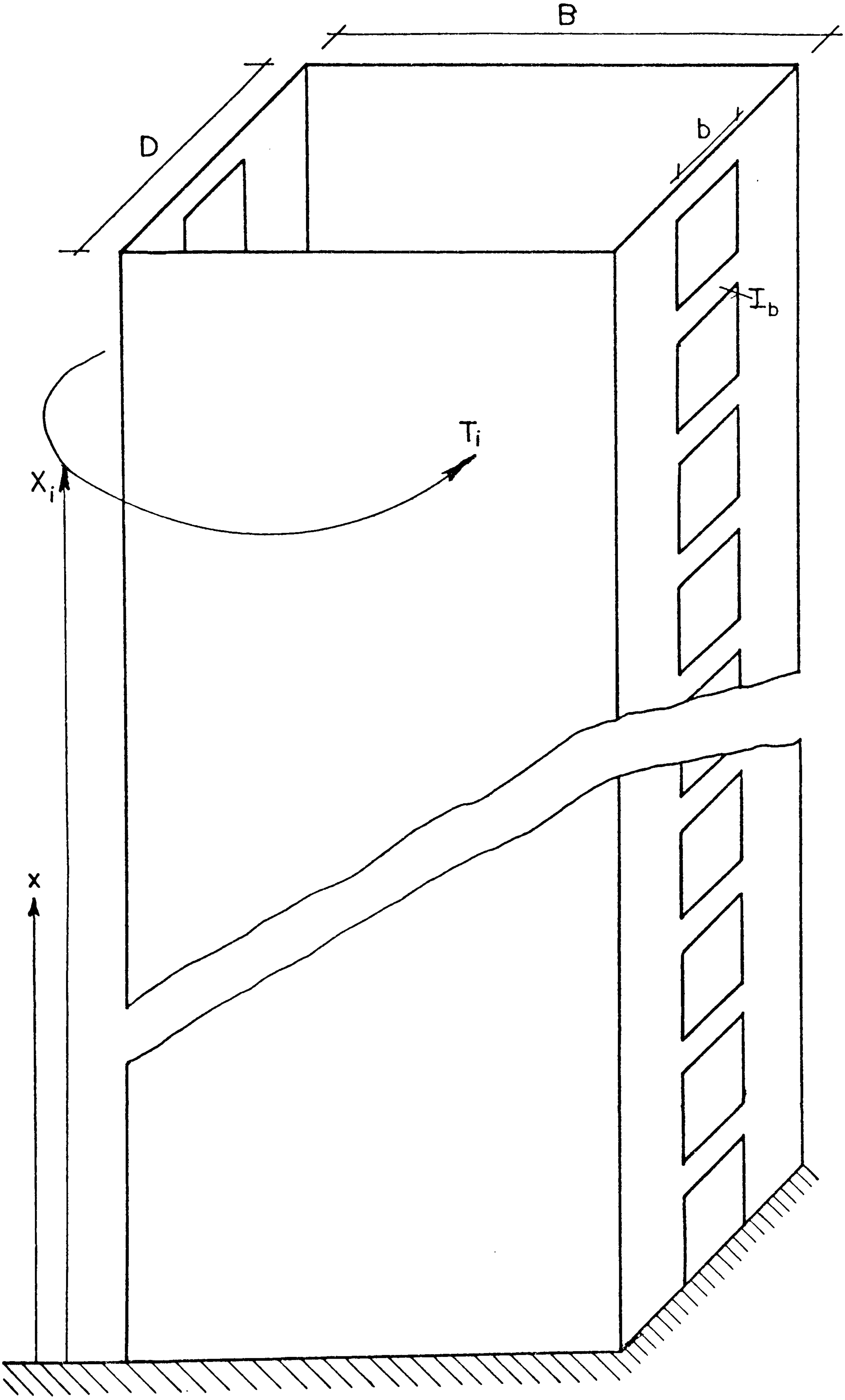


Fig. 3.7

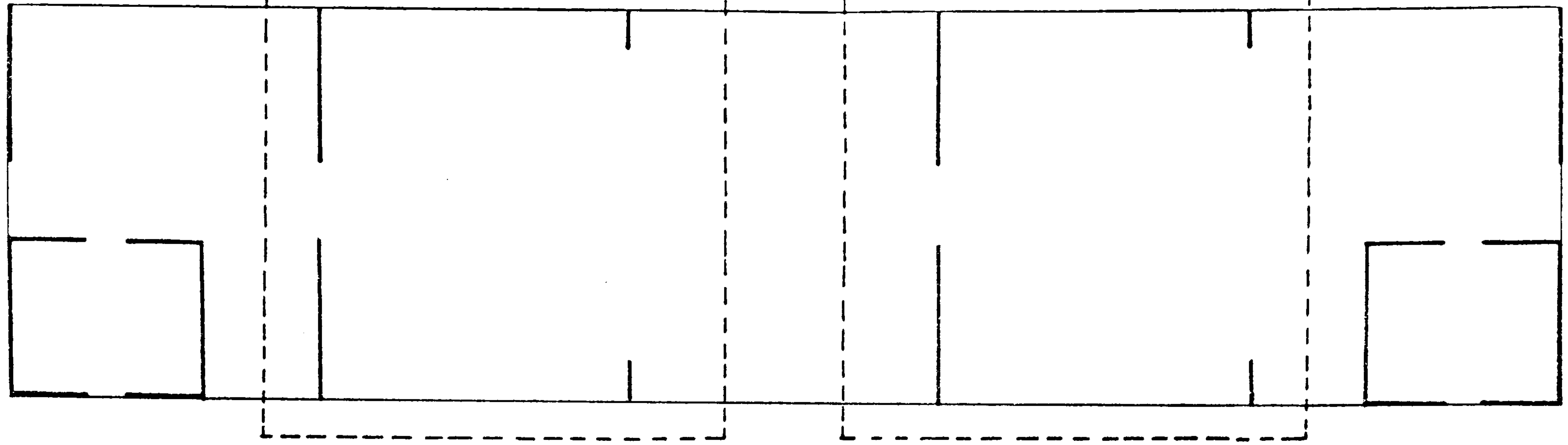


Fig. 3.8

STAGGERED SYSTEM OF MODEL 5

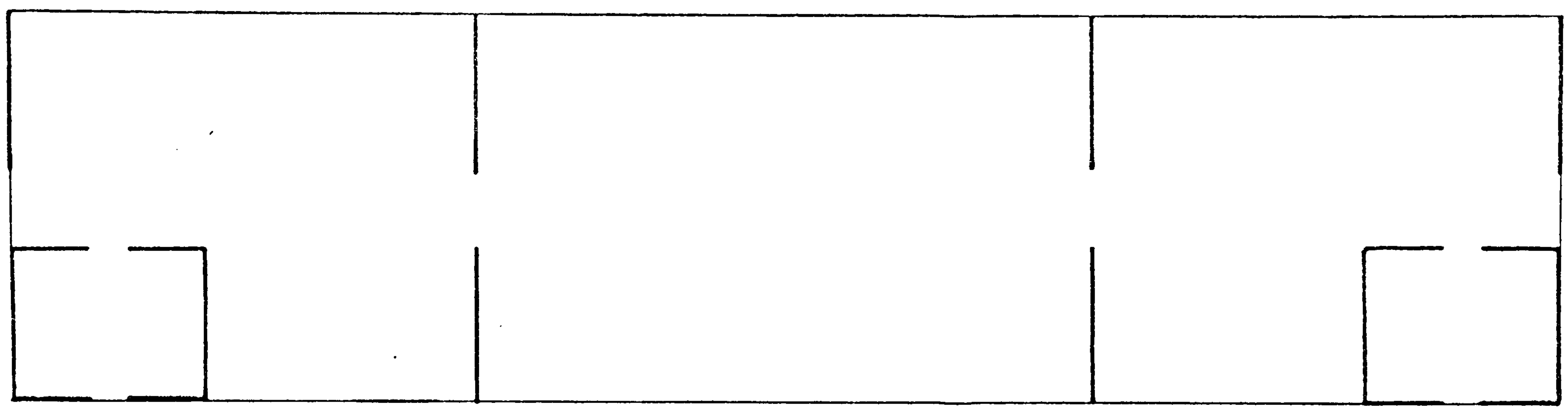


Fig. 3.9

SIMPLIFIED SYSTEM USING COUPLED SHEAR WALL UNITS

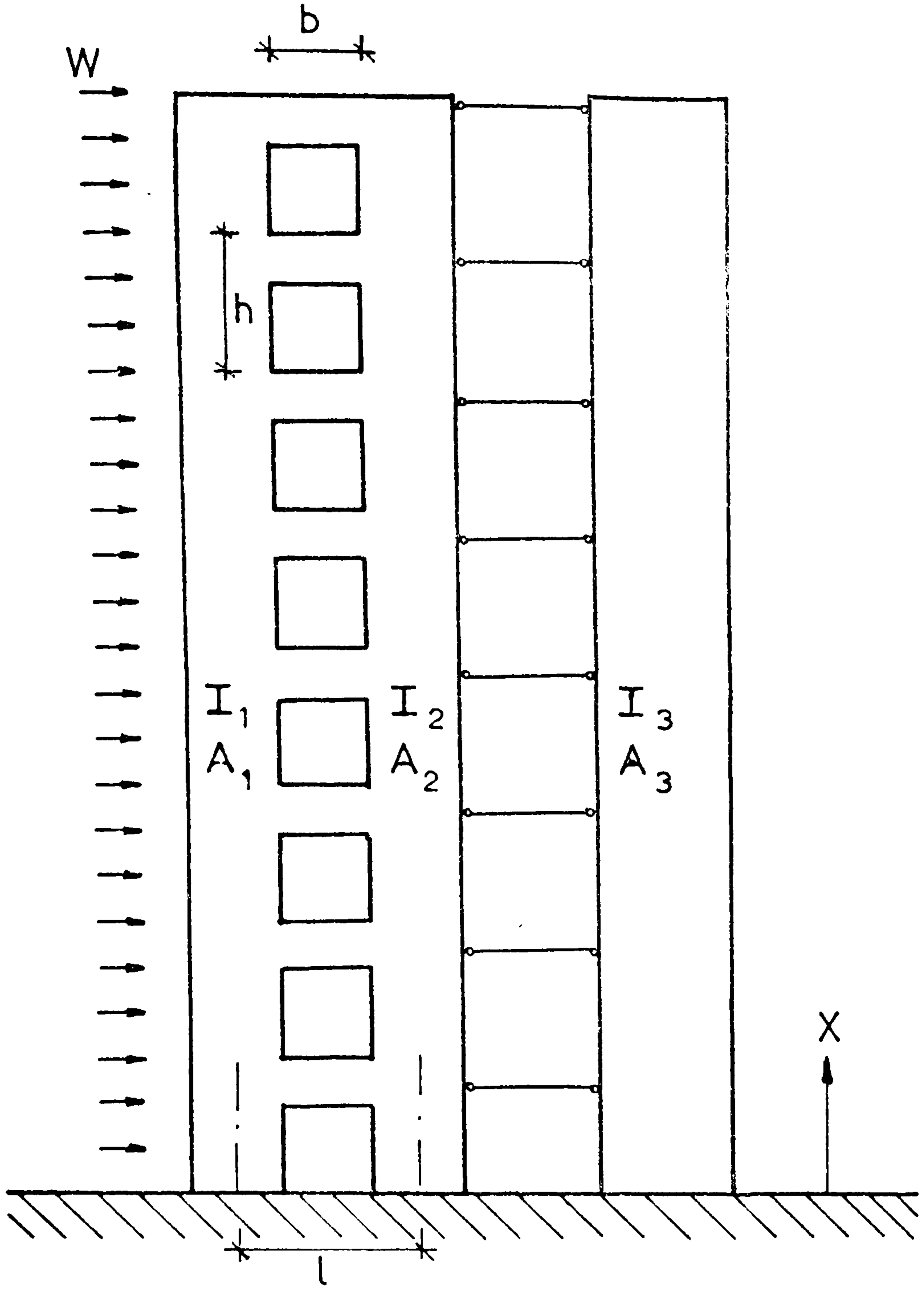


Fig.3.10

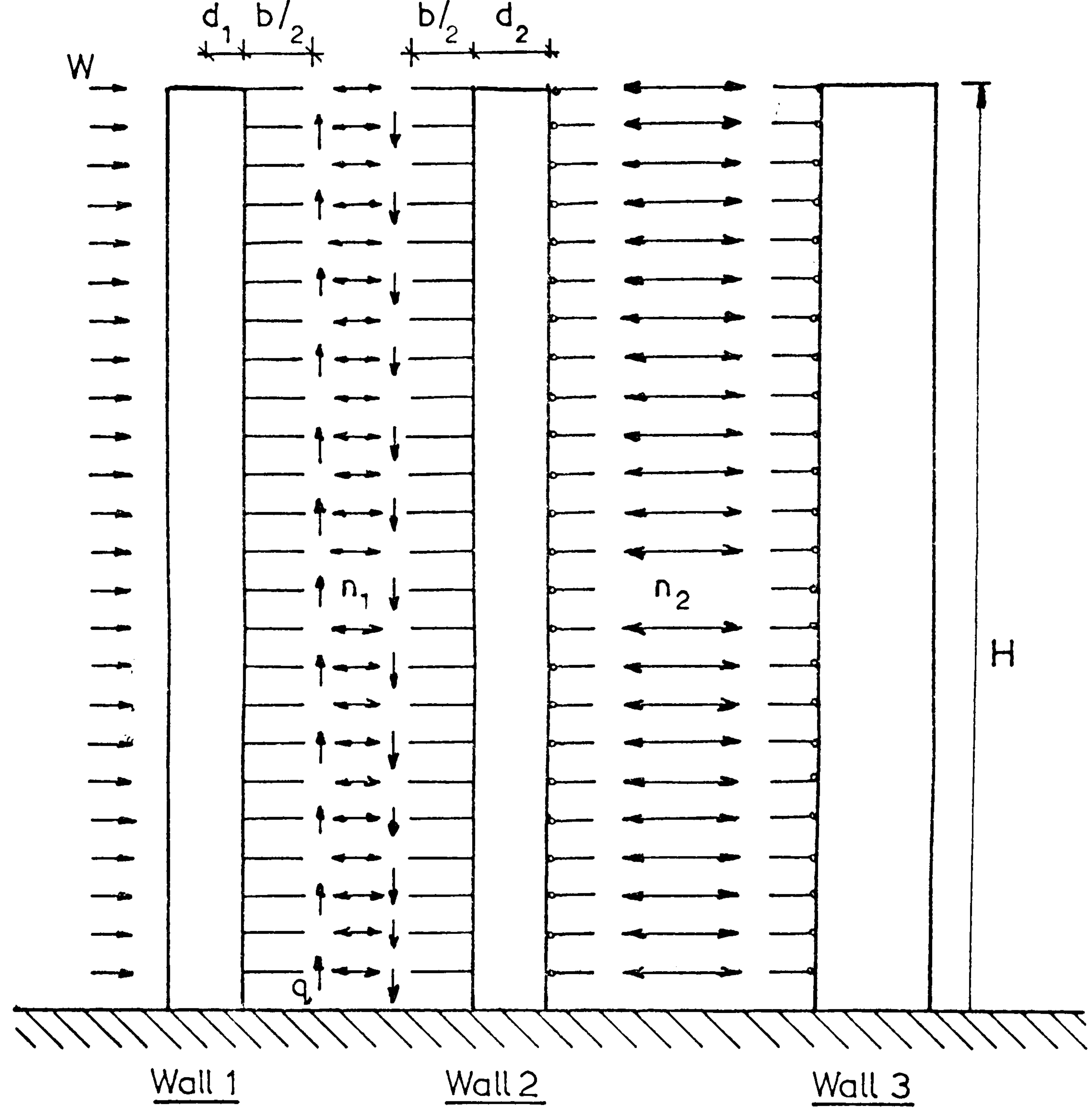


Fig.3.11

CHAPTER 4

4.1 INTRODUCTION

As indicated in Chapter 2 there is an acute shortage of published data on the behaviour of staggered-truss/wall beam systems. Experimental work on high-rise structures is largely restricted to elastic tests on small-scale metal or plexiglass models (16, 38, 45, 52, 55), and elasto-plastic tests on single-storey large scale units (30, 66). When only a short history of the performance of a structural system is known then data from elastic, static and dynamic, and elasto-plastic model tests can provide valuable information on which to assess a mathematical model of its behaviour. The possible effects of structural modifications and gaseous explosions can also be studied.

In view of this lack of experimental evidence several single-bay and multi-bay, three-dimensional models were constructed. Static and dynamic tests were carried out on the models to provide information for use in assessing theoretical analyses. Prior to this investigation there have been no published experimental results on models of complete structures comprising/

comprising staggered wall-beam systems, core, single and coupled shear-walls.

The model dimensions and material parameters are given for all models in the Figures and Tables of Appendix A.

4.2 PERSPEX MODELS

Two twenty-storey models were fabricated from sheet perspex, one 2-dimensional the other 3-dimensional. Model No. 1 was constructed by machining a 12.5mm perspex sheet to the desired twenty-storey profile and bolted at foundation level between two angle sections using 12.5mm diameter bolts, Fig. 4.1. Model No. 2 was constructed by slotting the floor slabs to accommodate the two vertical frame-bents, each of which was machined in one piece, and welded into position using TENSOL No. 7 cement. The assembly was then welded into a 300mm x 300mm x 25.4mm thick, perspex base-slab, Fig. 4.2.

4.3 MICRO-CONCRETE MODELS

The tendency to adopt shear-wall systems, and possibly staggered systems (25, 88), in earthquake prone areas necessitates a better understanding of their elasto-plastic behaviour. Data on such behaviour is very sparse (65) and largely restricted to theoretical analyses (27, 61). Micro-concrete models were therefore fabricated and tested, to provide some data on the ultimate load capacity and ductility of these complex structures in addition to providing information on their elastic, static and dynamic characteristics.

4.4 MIX-DESIGN

Before any construction could begin it was necessary to establish a concrete mix. To meet geometric similitude requirements micro-concrete would be produced by reducing the size of all constituent materials including cement by a scale factor. However, this simplified approach is impractical since scaled fine aggregate has excessive water requirements and more finely ground cement is not available. As a result of this the micro-concrete was designed as it would be for ordinary concrete. The/

The mix design criteria, which to some extent conflict are:-

a) The mix design should be highly workable, since it would be poured into 12mm thick walls.

b) Its initial setting time should permit adequate time for casting and any other manipulations required prior to setting.

c) The mix should simulate the characteristics of concrete, i.e. the stress/strain curve should be the same general shape as that of normal concrete.

d) Shrinkage should be minimal.

A program of mix design was undertaken with the water/cement ratio and cement/aggregate ratio as independent variables. Conflicting water requirements occur, since small amounts of water reduce shrinkage, but wetter mixes are easier to place. The natural gradation of local sands was used below a specified maximum size (1.5mm). This being mixed with whin chips (less than 6mm in size), from a local quarry. The maximum size of sand and whin/

whin was determined from the thickness of section being poured and the reinforcement spacing in the model. Whin (Dolerite) was used in preference to other materials because of its availability and very high compressive strength.

The cement used was ordinary Portland Cement and the water/cement ratio was varied from 0.34 to 0.5, the value finally adopted being 0.37. The whin chips were washed before being used. An allowance was made for the moisture/content of both the sand and the aggregate when computing the required quantity of water for the mix. The ratios used for whin:sand:cement were 2:1:1 and their particle size distribution is given in Table 1.

Initially formwork was constructed to enable a series of trial walls of varying thickness, (i.e. ranging from 15mm to 6mm) and dimensions, with trial mixes of varying aggregate types and gradings, to be investigated. (Plate 1). From the results of the tests carried out it was decided that 12.5mm thick walls were the most practicable and therefore the floor slabs and walls of the micro-concrete models were of this thickness.

4.4.1 VIBRATION EQUIPMENT

Some difficulty was encountered in obtaining an efficient and reliable vibration system for casting the small sections used in the models. The form-work was clamped to a vibrating table which operated at frequencies up to 50 Hz at 5g, this was supplemented by a poker vibrator of small amplitude and higher frequency and four small pneumatic "Ball-Type" vibrators.

4.4.2 REINFORCEMENT DETAILS

Wire mesh reinforcement was used within the walls, whilst the floor slabs had longitudinal steel supplemented by the wire mesh. The mesh selected for all sections was 12.5mm x 12.5mm ($\frac{1}{2}$" x $\frac{1}{2}$") square mesh 19 S.W.G. giving 0.26% steel in each direction. This is consistent with other researchers (1), and corresponds with minimum code requirements for steel to control temperature and shrinkage cracks. Additionally it is commercially available and is ideally suited to these units by its regular shape.* Due to the thin slab sections the longitudinal steel was restricted to approximately 1.5mm diameter and No. 16 S.W.G. steel was used.

---

* Hexagonal mesh reinforcement was also tried, but proved less suitable.

Typical S.W.B. unit and slab reinforcement details are shown in Fig. 4.3. In models 4, 5 and 6 the steel in the walls was continuous from anchorage within the base reinforcement throughout the height of the structure to roof level and reinforcement in each floor slab attached to the vertical reinforcement using soft "steel-fixing" wire. Soldering between vertical and horizontal reinforcement was avoided to prevent any artificial rigidity of the complete wire-mesh cage.

Positioning of the steel within the slabs and wall units was difficult and in general when the micro-concrete was poured and vibrated the mesh tended to drift slightly off centre. This was reduced in later models by stretching the wire-mesh.

4.5 FABRICATION OF MODELS

Two methods developed for fabricating single-bay or multi-bay structural models are described in detail in the following sections. The first method is suitable for single-bay structures only whilst the second is suitable for either type.

4.5.1 MODEL NUMBER 3

The technique used in fabricating this twelve storey model was as described below.

It was intended to cast the S.W.B. units and floor slabs of this model individually, and after curing assemble the component parts into a single-bay twelve storey model. The floor slabs and wall-beam units were cast flat, finally using tapered perspex blocks with 12mm radius corners, (the edges of which had been smeared with petroleum jelly) to form the wall openings after several alternatives such as wooden and collapsible had been tried. (Plate 2). The wooden blocks tended to create re-entrant corners and swelling when moist, thus inducing cracking of the concrete. In addition they had a high frictional resistance along the edges making their removal potentially harmful. The collapsible block (see Fig. 4.4) had a limited success, but involved too much machine work and handling to be practicable.

The micro-concrete was made according to the mix design given previously and compacted into the formwork, which had been varnished to avoid absorption of/

of water from the mix, and given a light coating of release oil. After twenty-four hours the formwork was stripped. The peripheral timber was easily removed, but the perspex blocks had to be removed slowly and carefully to avoid cracking. This was achieved by tapping two small holes in each block and inserting bolts which were slowly tightened onto the base of the formwork. Subsequently the blocks were lifted vertically until clear of the model and then removed. The S.W.B. units were cured under wet hessian for twenty-eight days. The floor slabs were cured in a humidifier for forty-eight hours, and for a further twenty-six days under wet hessian. After curing, the component parts of the model were assembled. The floor slabs and wall-beam units were joined together using Araldite (adhesive) combined with the threaded reinforcement. (Fig. 4.5).

A 400 x 500 x 200mm deep concrete base with a wire mesh reinforcement cage and four 12.5mm diameter threaded studs extending through the top and bottom was then cast around the completed twelve storey one-bay model. (Fig. 4.6).

For/

For simplicity in construction, no lintel beams were provided and the only connecting media between vertical elements were the horizontal slabs.

4.5.2 MODEL NUMBER 4

The method of construction detailed in section 4.3.3 is suitable for single-bay structures but not for multi-bay structures. Since one purpose of the experimental investigation was to produce evidence for comparison with the analytical displacements and rotations of entire structures comprising staggered-wall-beam frame bents, cores, shear-walls and floor slabs, a method was developed to fabricate such complex models.

A trial attempt to cast columns, wall-beams, floor slabs and base monolithic in a single-bay eleven-storey structure resulted in Model Number 4. The method as developed is described below.

For each storey a polystyrene mould was cut and shaped to the plan and elevation specifications using an electrically heated wire. The required number of moulds were then positioned inside an 18mm. plywood channel form and held in position using tie-rods/

rods as shown in Fig. 4.7. When cast into the formwork around the moulds the concrete formed a complete three-dimensional structure comprising wall-beams, columns and slabs. Considerable difficulty was incurred during pouring since access to the areas requiring concrete was very limited and the vibration techniques were inadequate to give the necessary compaction and penetration, resulting in the formation of air-pockets within specific areas. After a twenty-four hour period the shuttering was stripped and prior to curing the polystyrene was removed and repair work carried out on the areas where the concrete had failed to penetrate. The material used for the repair work was a mortar made from a fine-grain sand, cement and water. Most of the incomplete sections were in the floor slabs and in some cases were extensive as can be seen in Plate 3.

4.5.3 MODELS NUMBER 5 AND 6

The degree of success of Model No. 4 justified further development of the method of fabrication and led to two further models both five bays wide and consisting of wall-beams, coupled shear walls, cores and floor slabs. As in the previous models lintel beams were omitted.

The/

The first two attempts were unsuccessful due to the lack of penetration of the concrete, lifting and relative movement of the polystyrene moulds within the formwork (Plate 7). Modifications to the water/cement ratio to give better workability, and increase in the floor-slab thickness to 15mm ($^5/8"$) allowing better penetration, a closer supervision of aggregate sieving and an improved system of clamping down the moulds (see Fig. 4.8) produced Models Number 5 and 6. Model 5 did not require any remedial repair work and had an average variation in cross-section of under $\pm$ 5%, Model 6 required some repair work where the base had moved relative to the walls as mentioned below. The base of these models, as with number four, was cast simultaneously as an integral part of the structure. Model 4 had four 12.5mm diameter threaded studs cast in the base to enable a secure fixing to the test rig, the larger models had eight small conduits cast into the base to allow 12.5mm diameter threaded rod to pass through and anchor the models firmly on the rig (Fig. 4.9).
Rotation of the base slab on Model 6 occurred during casting, resulting in some concrete escaping from the formwork and the conduits being misaligned with the holes of the test rig. After curing, 'Unifix Anchor Bolts' were driven into the slab which enabled the model to be bolted to the test rig and/

and additional concreting carried out on the base. The underside of models 5 and 6 and the surface cf the test rig were coated with a layer of industrial grease to increase the manoeuvrability of the models on the test rig and ensure their correct position.

4.6 TESTING EQUIPMENT AND PROCEDURES

A heavy steel rig was constructed to facilitate both the static and dynamic experimental investigations. The base of the rig consisted of a 25.4mm thick steel plate supported on three steel 305 x 102 x 46.8kg channel sections. Three similar channel sections were welded upright to the base and supported another 25.4mm thick steel plate, which provided an inflexible vertical support. The steel plates were drilled with 13.0mm diameter holes on a 200mm square grid (Fig. 4.10).

4.6.1 STATIC TESTS

Model Number 1, bolted between the angle sections mentioned in section 4.2 was secured by four 12.5mm diameter bolts, to the vertical steel plate of the test rig. In order to restrain the horizontal plane-frame to vertical movements only, a small roller support was made and positioned as shown in Fig. 4.11.

The/

The perspex base of model number 2 was fastened directly to the horizontal steel plate by means of a 12.5mm thick steel plate section and four 12.5mm diameter bolts as shown in Fig. 4.12.

Models 3 to 6 were bolted directly on the test rig in the manner shown in Fig. 4.13, 4.14 and 4.15 using the threaded studs which had been cast into the base in the case of Models 3 and 4, threaded rod passing through the conduits in the case of model 5 and the unifix anchor bolts fixed into the slab of model number 6.

All models were subjected to a series of distributed and point lateral loads. These loads were applied at the mid-span points of the floor slabs by means of dead loads hanging from standard wire-ropes in the case of models 3 and 4, and nylon line for model number 2, looped around the floor-slabs and passing over free running pulley. Dead loads were applied directly on nylon line held in position with P.V.C. tape on Model No. 1. Distributed lateral loads were simulated by simultaneously applying point loads at intermediate levels of the structures as indicated in Appendix B. The pulleys were supported between two sloping members with adjustable pulley positions (Fig. 4.16).

Models/

Models number 5 and 6 were tested in a similar manner with a system of pulley wheels supported on an adjustable sloping frame (Fig. 4.17) which permitted varying positions and eccentricities of loading. These two models were subjected to a centrally distributed lateral load, a central point lateral load at the top and similar loads at an eccentricity of 160mm off centre.

Lateral movements of the structures were recorded at various levels on a series of dial gauges of $\pm$ 0.01mm accuracy. Several dial gauges were also positioned at foundation level to check for any base movement. A network of 120ohm electrical resistance strain gauges, (manufactured by SHOWA), attached to walls and columns of the models measured the micro-strain during loading and were monitored on a Peakel portable strain-indicator and extension box of ten gauge capacity. Before these gauges could be applied it was necessary to smooth out irregularities on the concrete surfaces. This was done first with a small grinding wheel and completed with fine emery paper. Industrial alcohol was used to remove fine dust particles and the surfaces given a thin layer of water-proof adhesive on which to apply the gauges.

The/

The adhesive used for the gauges was Eastman 910' with a catalyst and they were covered with P.V.C. tape to isolate them from atmospheric conditions. Dummy strain gauges were also mounted on both acrylic from the original sheets as those from which the models were constructed, and micro-concrete slabs and beams cast at the same time as the other units.

All static tests were conducted under a strict time schedule to minimize the effects of creep and repeated three times for each load range. Where the results were inconsistent the test procedure was repeated until the results obtained were considered reliable.

4.6.2 DYNAMIC TESTS

With the models fixed to the test rig as for static tests, a solenoid actuator bolted to the vertical steel plate of the test rig and driven by a variable frequency generator subjected models number 3, 4, 5 and 6 to forced vibrations of differing amplitudes and frequencies. Two small Environmental Equipment accelerometers, Type AR 40 were/

were used to monitor the response of the structures and check the actuator output signal. Output from the accelerometers was amplified and viewed on an accurately calibrated, high-gain, two-channel storage oscilloscope and registered on an ultra-violet recorder (Plate 8).

An initial approximate detection of natural frequencies was found from the response of the structure as it was swept through a range of frequencies. Once approximate values of natural frequency had been detected, interest was focused around these frequencies, where the structure was forced to vibrate at smaller increments until a particular natural frequency was identified. After having established a natural frequency an accelerometer was positioned at each level in turn and the amplitude of the response wave recorded on the oscilloscope and U.V. recorder, giving the mode shapes. An estimate of damping present was made using the half-power bandwidth technique described in Appendix D.

The/

The equipment available restricted the determination of mode shapes to the concrete models since the mass of the accelerometers was comparable with that of the perspex models. The fundamental natural period, and hence frequency, of models number 1 and 2 were found using the free vibration response record from pluck tests on the models. This response was monitored from the output of selected strain-gauges, which was amplified through an S.E.4000, a.c. amplifier system and used as input to a six-channel ultra-violet recorder. Estimates of damping were obtained using the logarithmic decrement of the response curve on the U.V. records as described in Appendix D.

Resonance testing was also carried out on the combined test-rig and activator system to establish any natural frequencies within the range of model testing.

4.6.3 ULTIMATE LOAD TESTS

In the ultimate load tests Models 3, 4 and 5 were subjected to a steadily increasing, lateral point load on the centre-line at the top of the models. Dial/

Dial gauge readings only, were taken since the strain gauge readings were of little value after the onset of cracking and were unreliable after the dynamic testing. At failure, the mode of collapse was noted.

Originally the intention was to conduct a cyclic test on Model Number 5 to assess the ductility characteristics of the structure. This was not possible since flexural cracking and subsequent failure occurred on the first cycle. There were no signs of shear failure and since the only vertical loading was self-weight, it is possible that the applied loading was too severe. The loading on Model Number 6 was applied at level 11 thus reducing the magnitude of the bending moment and hopefully inducing more useful cracking. This did not happen and the failure pattern of Model Number 5 was repeated.

The failure modes of these models are discussed more fully in Chapter 6.

4.6.4 TORSION TESTS ON MODEL NUMBER 2

A small test-rig was available for torsion tests on Model Number 2, this is shown in Fig. 4.18 and Plate 4. The model was bolted to the base of the rig as in other tests and a circular, perspex jig was machined to enable the application of a pure torsional moment at any level. The twisting moment was applied by means of dead loads suspended from nylon threads passing over pulleys and subjecting the jig to equal and opposite tangential forces. The deflections of the floor-slabs were measured by dial gauges positioned on opposite corners of the floor-slabs at various levels. (Fig. 4.19). The results, presented in matrix form in Appendix B, give the measured displacements of selected floor-slabs for four positions of applied unit twisting moment.

4.7 EVALUATION OF MATERIAL PARAMETERS

To provide values for material properties which could be used in the theoretical analyses of the models, samples from the original perspex sheets, reinforced slabs (400mm x 300mm x 125mm), rectangular beams (50mm x 25mm x 400mm long) and unreinforced/

unreinforced cylinders (300mm x 150mm dia.) were subjected to static and dynamic tests. The compressive strength found from crushing cylinders had an average value of $55N/mm^2$ after sixty-four days.

Static beam tests were carried out on the rectangular beams and slabs which were strain gauged top and bottom, normal and parallel to their major axis. The beams were simply supported on knife-edge supports, loaded at their quarter spans and their central and support deflections measured using dial gauges. Results of these tests gave an average of the static value of modulus of elasticity and Poisson's Ratio. The beams cast at the same time as Model Number 4 cracked when curing. The load/deflection graph was therefore drawn after crushing a cylinder, and the initial tangent modulus taken from the resulting curve was assumed to estimate Young's Modulus.

Since the variation of modulus of elasticity with frequency of vibration is very complex, the value found at the first natural frequency of a vibrating cantilever is given. (Appendix C).

AGGREGATE		SAND	
SIEVE SIZE (mm.)	% PASSING	SIEVE SIZE (mm.)	% PASSING
6.35	100	1.2	100
4.76	65	0.6	93
2.4	17.8	0.29	22
0.6	3.0	0.15	3.5

PARTICLE SIZE DISTRIBUTION TABLES FOR SAND AND AGGREGATE

Table -1

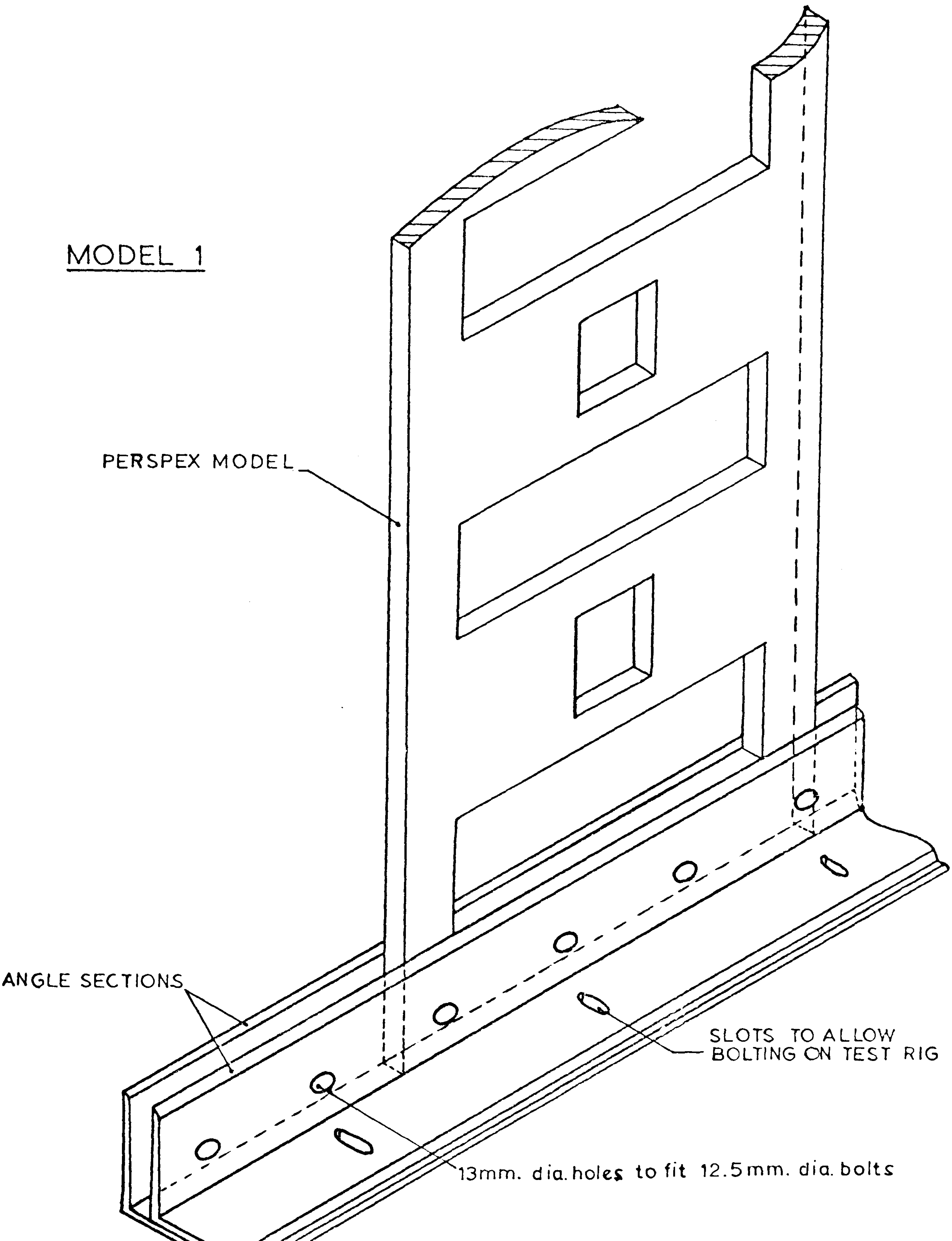

MODEL 1
PERSPEX MODEL
ANGLE SECTIONS
SLOTS TO ALLOW
BOLTING ON TEST RIG
13mm. dia. holes to fit 12.5mm. dia. bolts

Fig. 4.1

MODEL 2

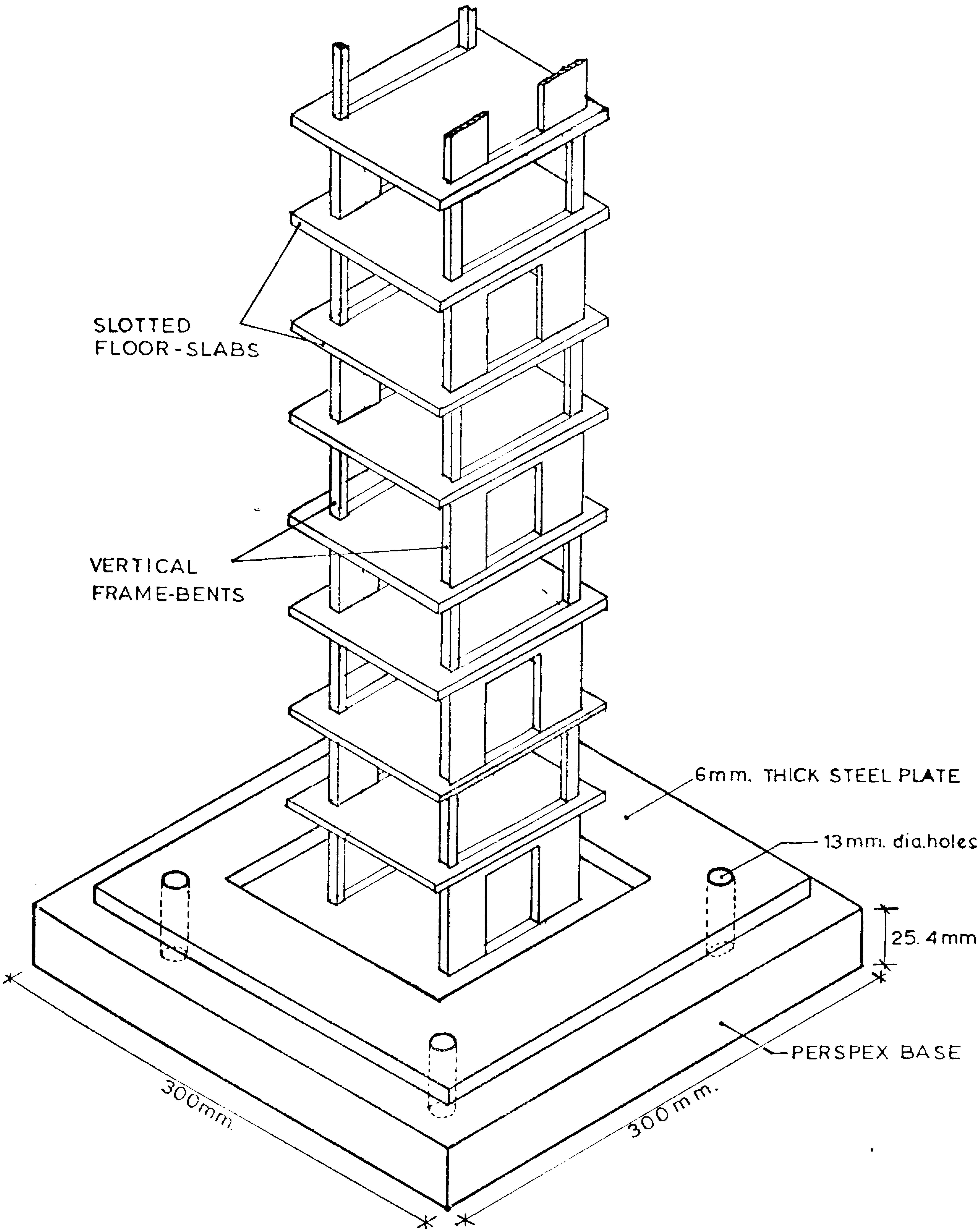
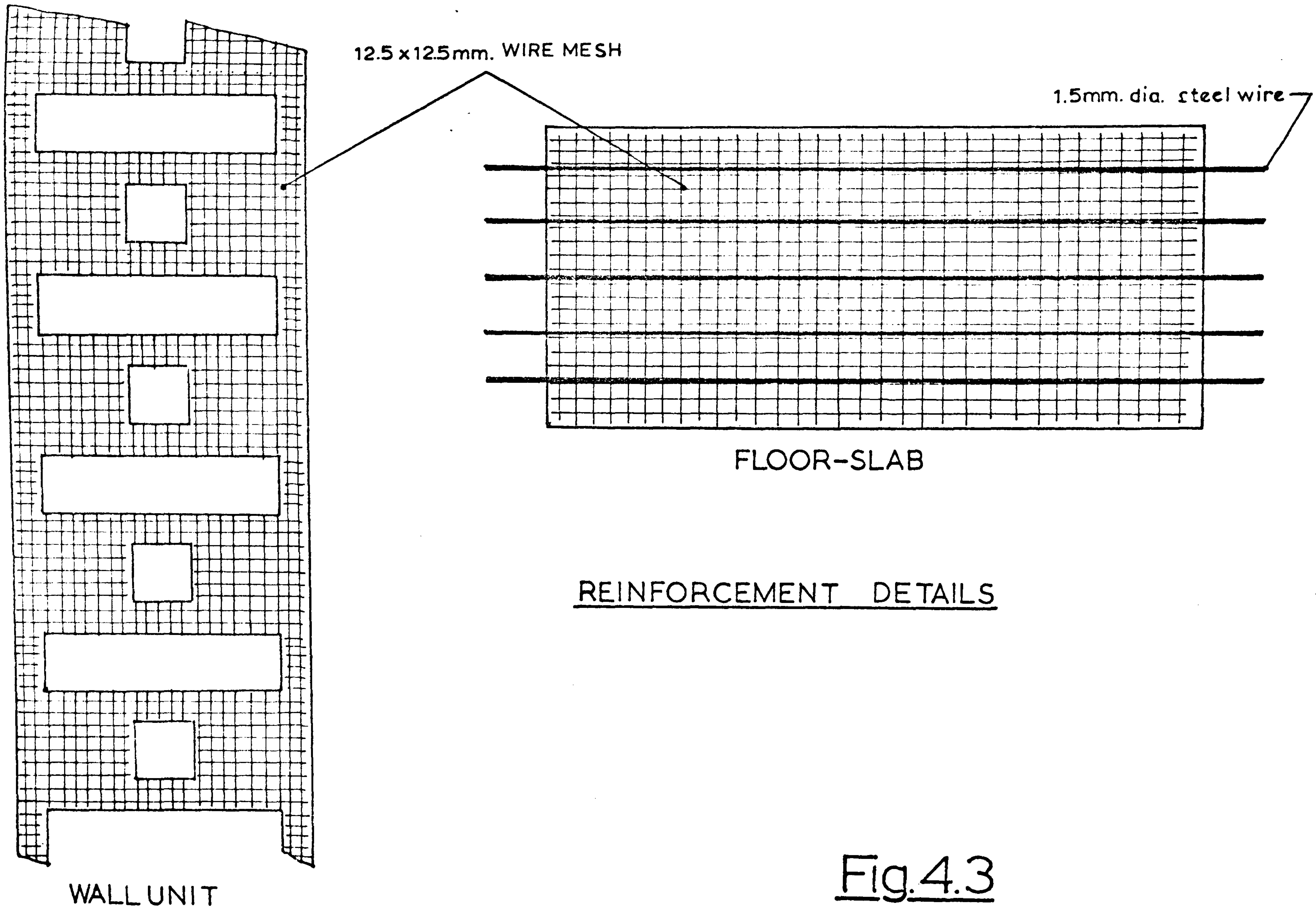


Fig.4.2



REINFORCEMENT DETAILS

Fig.4.3

COLLAPSIBLE — PERSPEX BLOCK

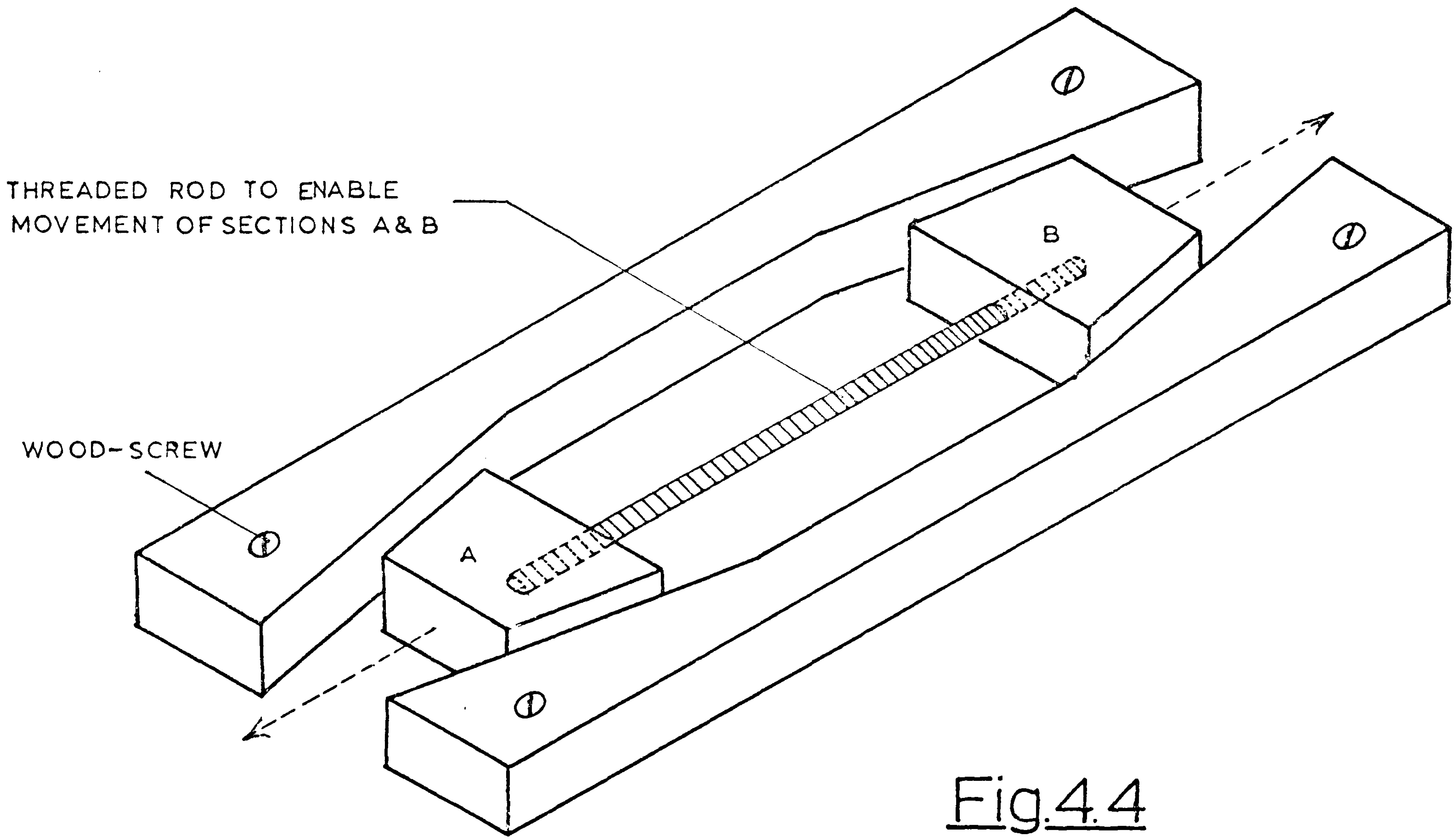


Fig.4.4

FABRICATION OF MODEL 3

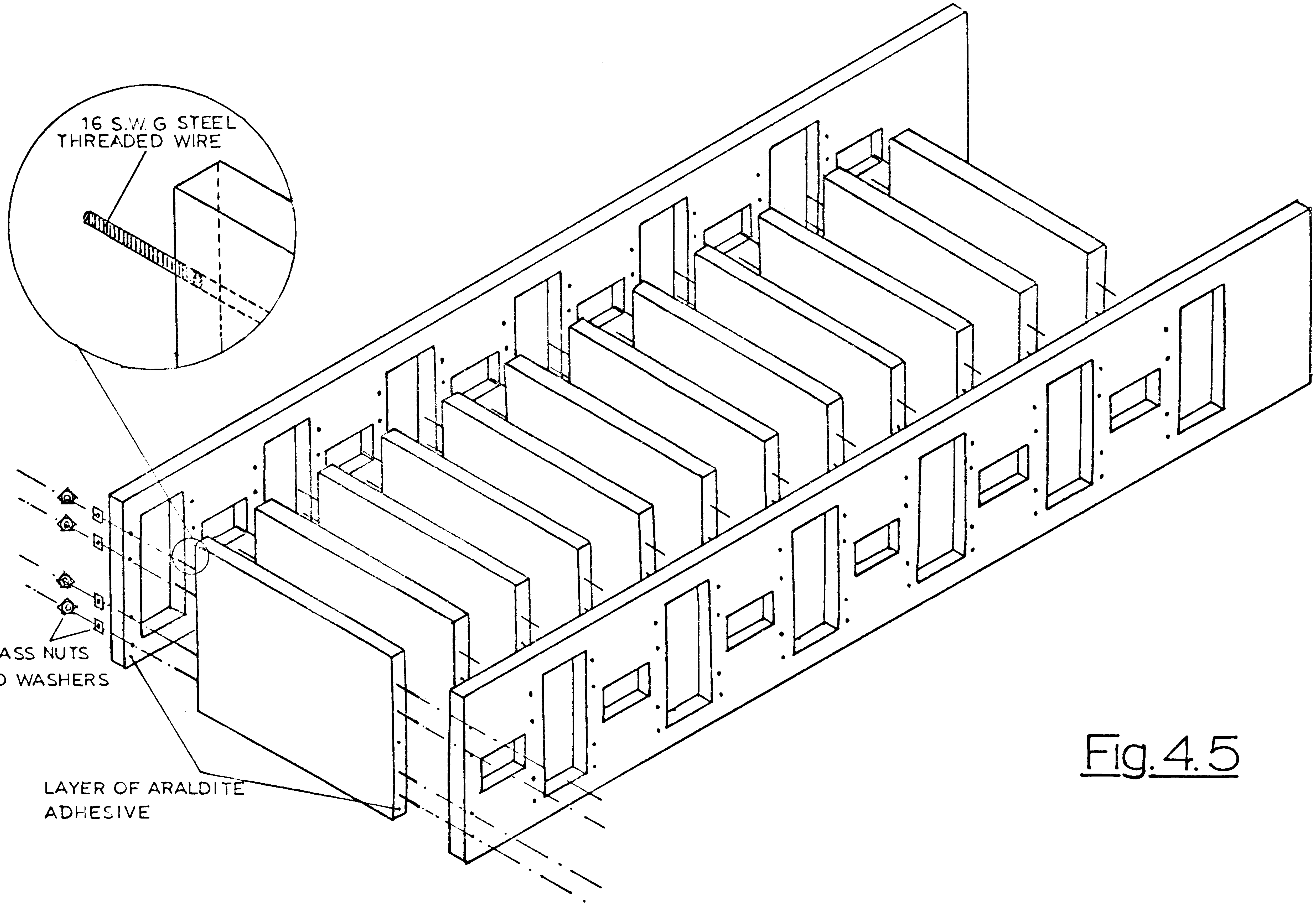


Fig. 4.5

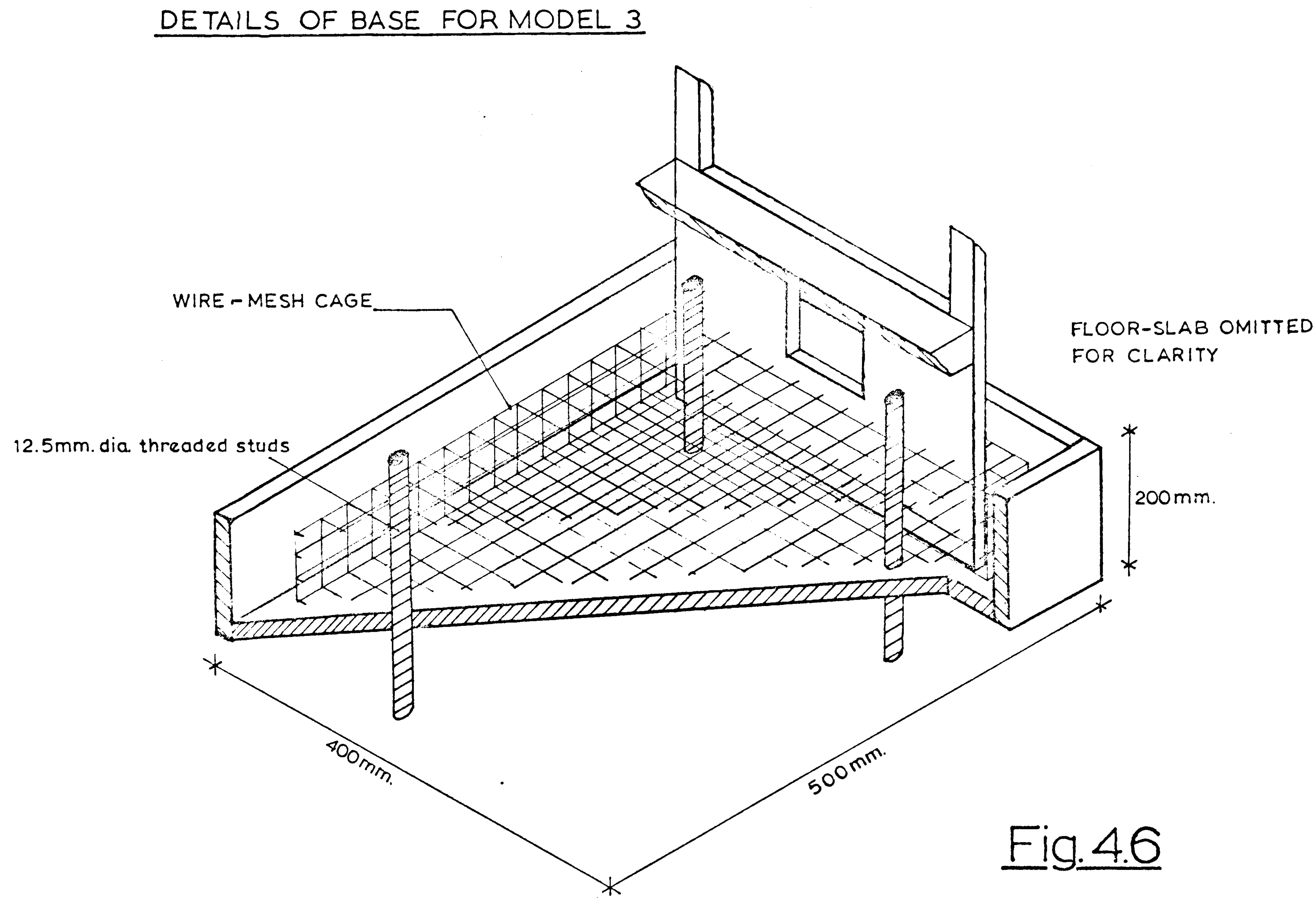

DETAILS OF BASE FOR MODEL 3
WIRE-MESH CAGE
12.5mm. dia. threaded studs
FLOOR-SLAB OMITTED FOR CLARITY
200mm.
400mm.
500mm.

Fig. 4.6

FORM, MOULDS AND REINFORCEMENT
FOR MODEL 4

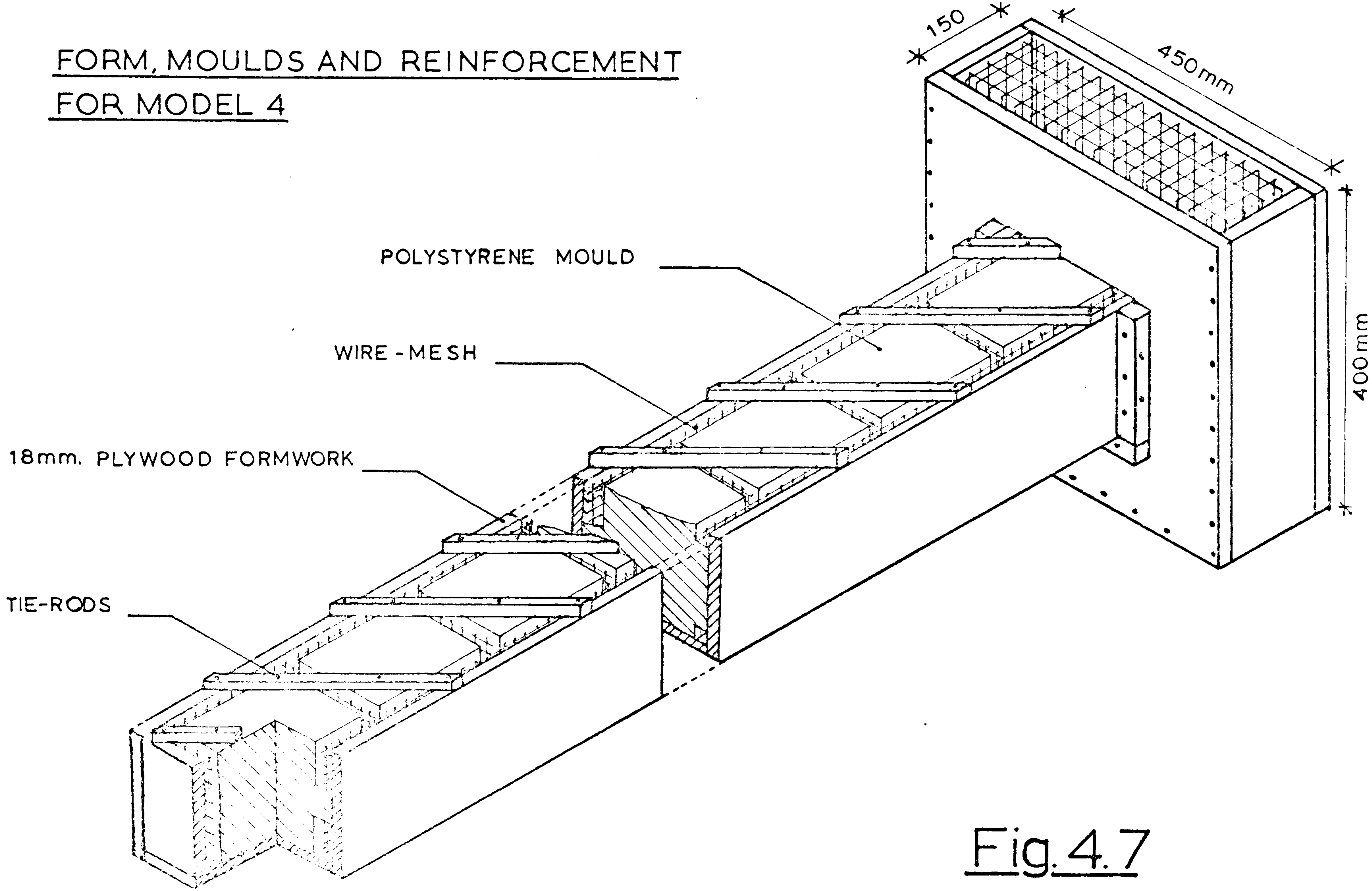


Fig. 4.7

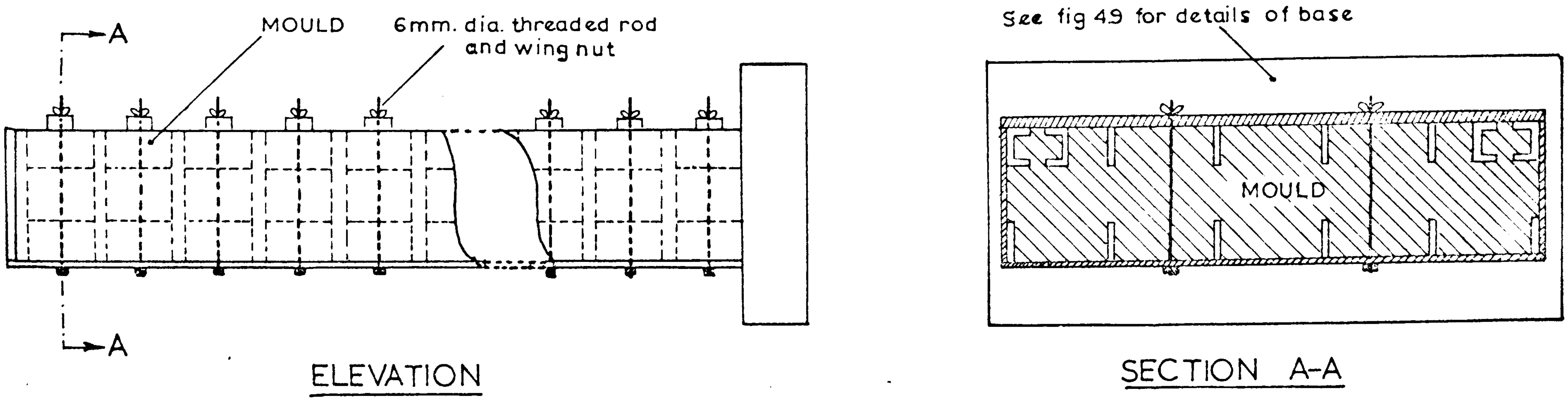

MOULD
6mm. dia. threaded rod and wing nut
A
A
ELEVATION
See fig 4.9 for details of base
MOULD
SECTION A-A

Fig.4.8


PART SECTIONAL ELEVATION
15mm. dia. hydraulic piping
12.5 x 12.5mm. wire-mesh
850mm.
200mm
200mm
200mm
500mm.
200mm.
END ELEVATION

Fig.4.9

TEST RIG

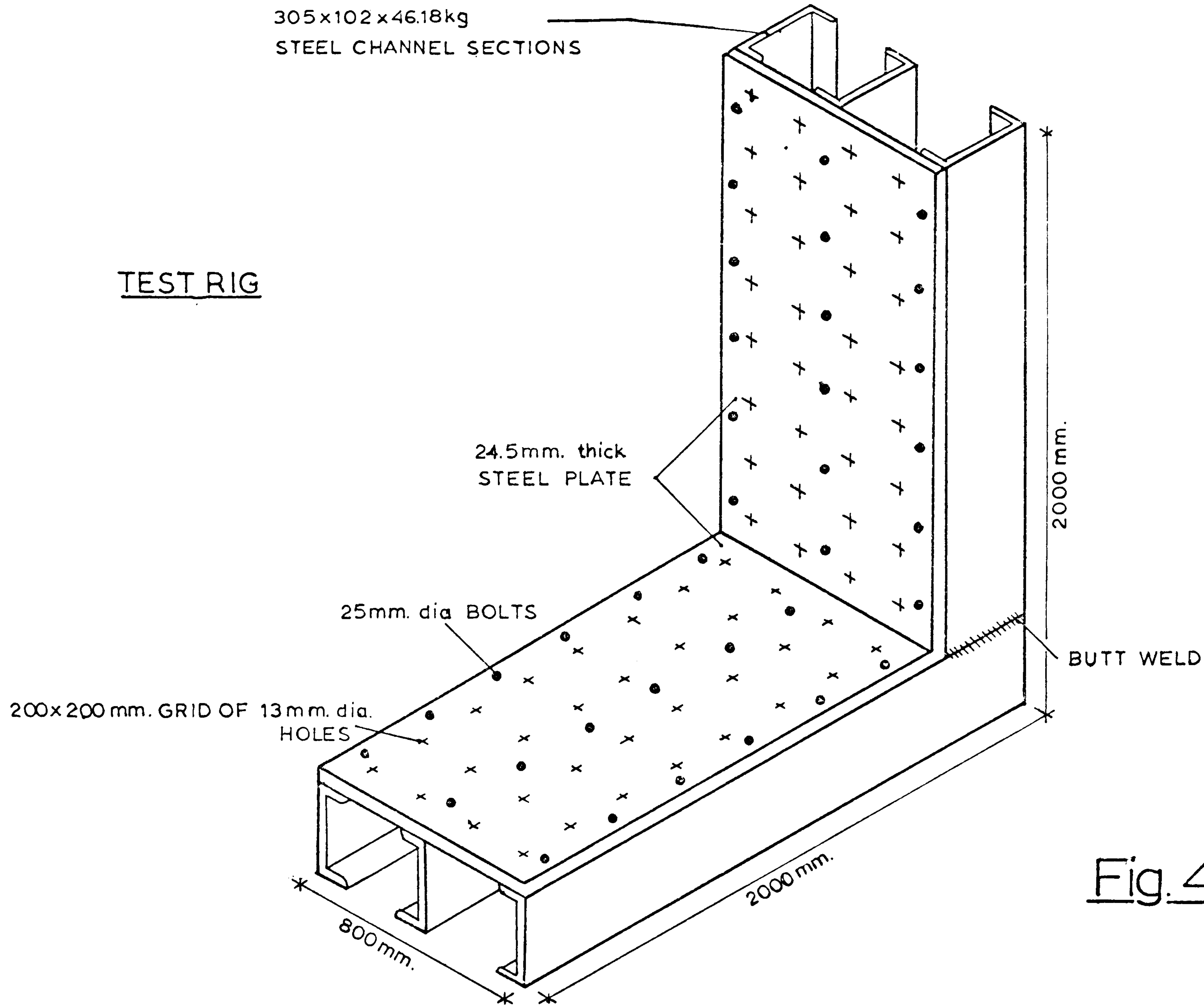


Fig. 4.10

ROLLER SUPPORT TO PREVENT LATERAL MOVEMENT OF MODEL 1 DURING TESTING

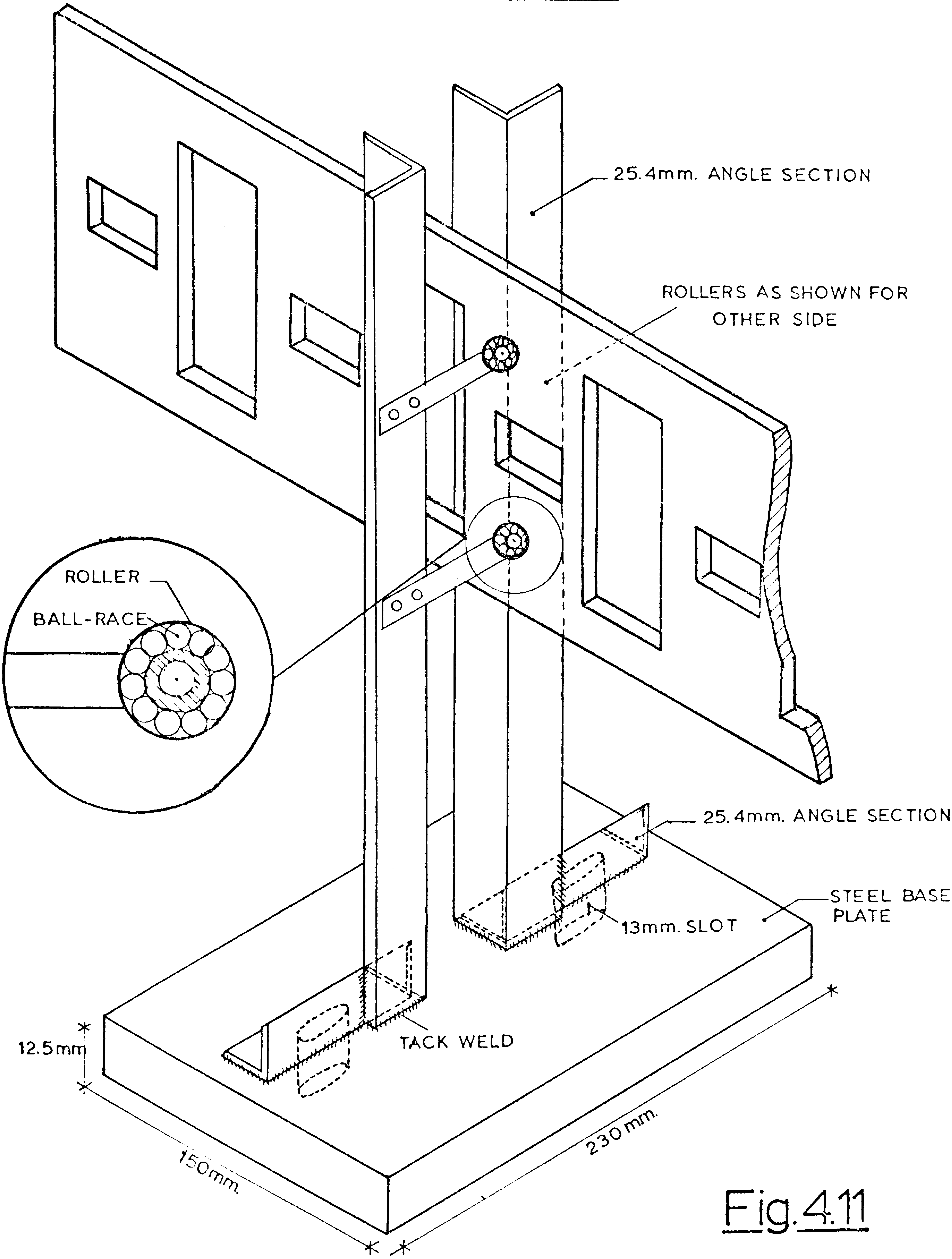


Fig.4.11

BOLTING OF MODEL 2 ON TEST-RIG

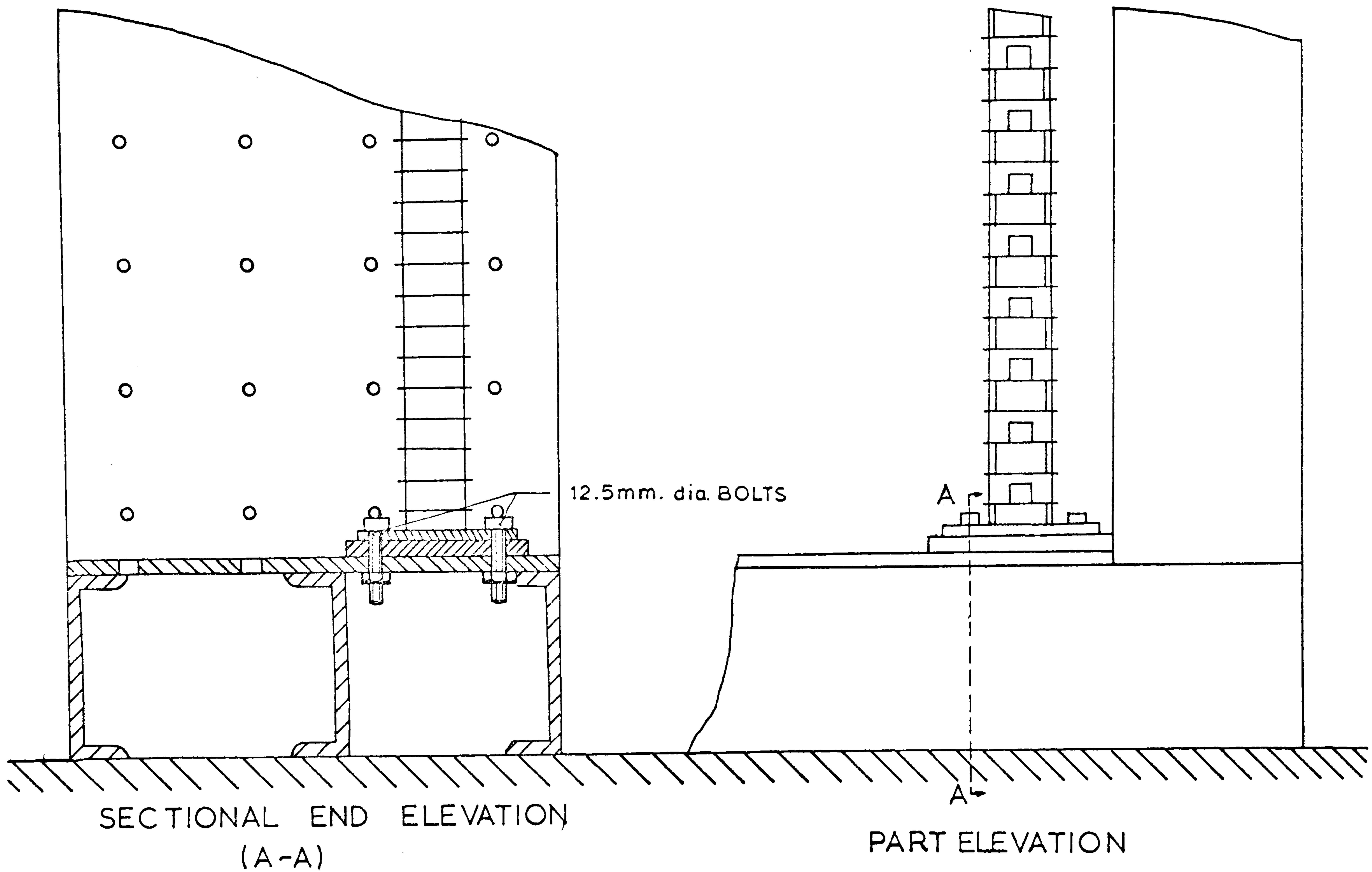
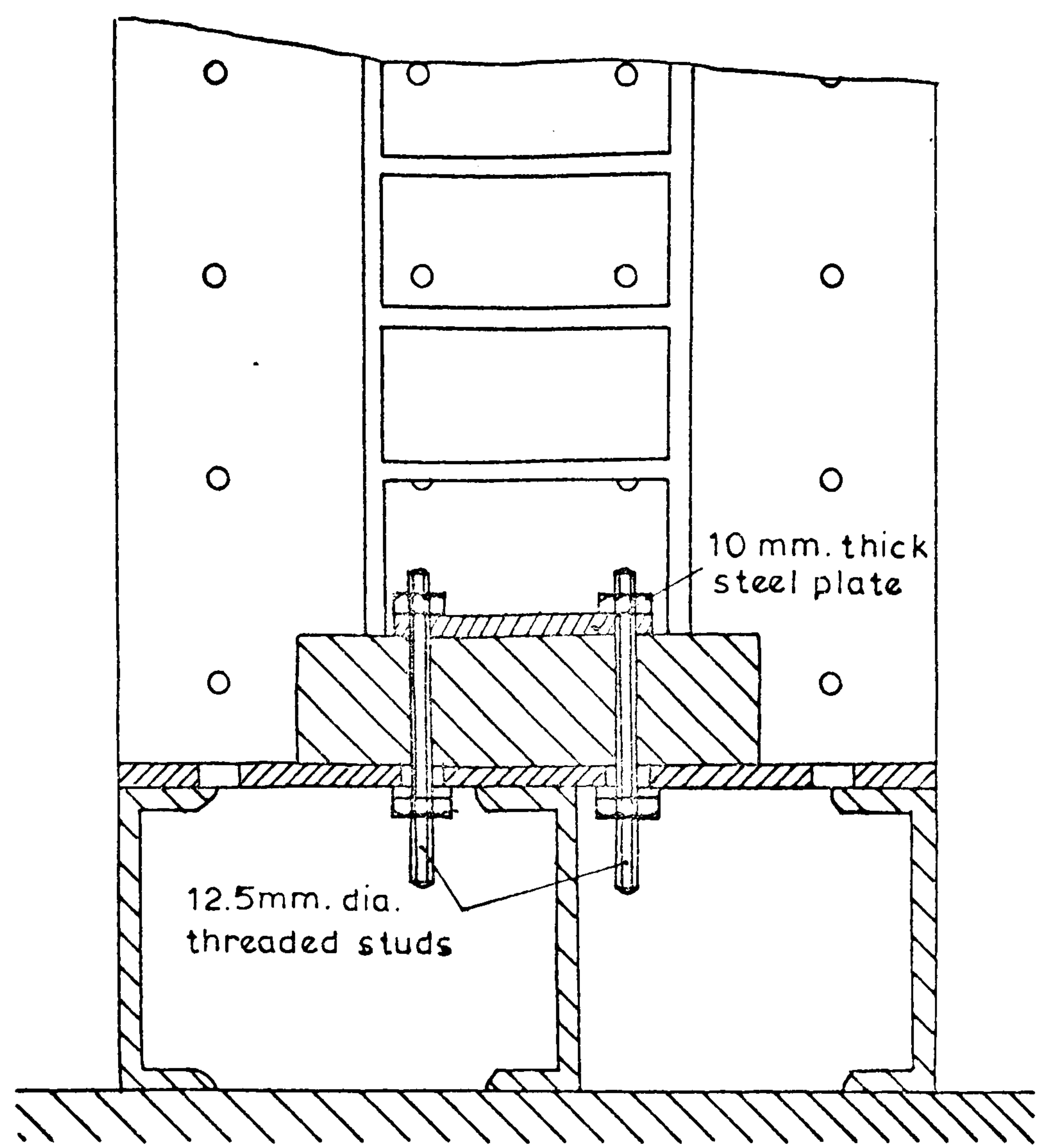


Fig.4.12

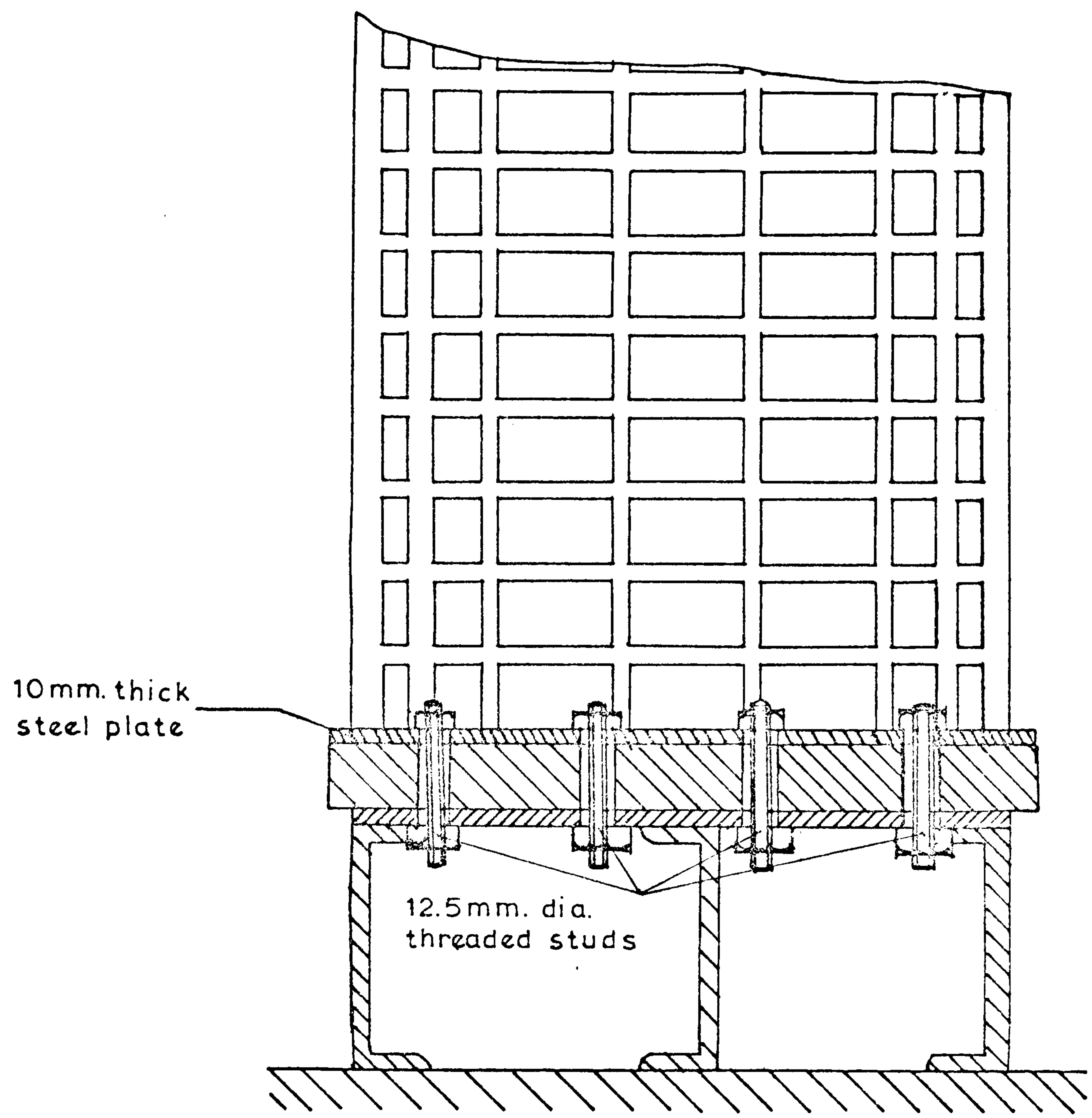
BOLTING OF MODELS 3 & 4 ON TEST-RIG



SECTIONAL END ELEVATION
AS IN FIG. 4.12

Fig. 4.13

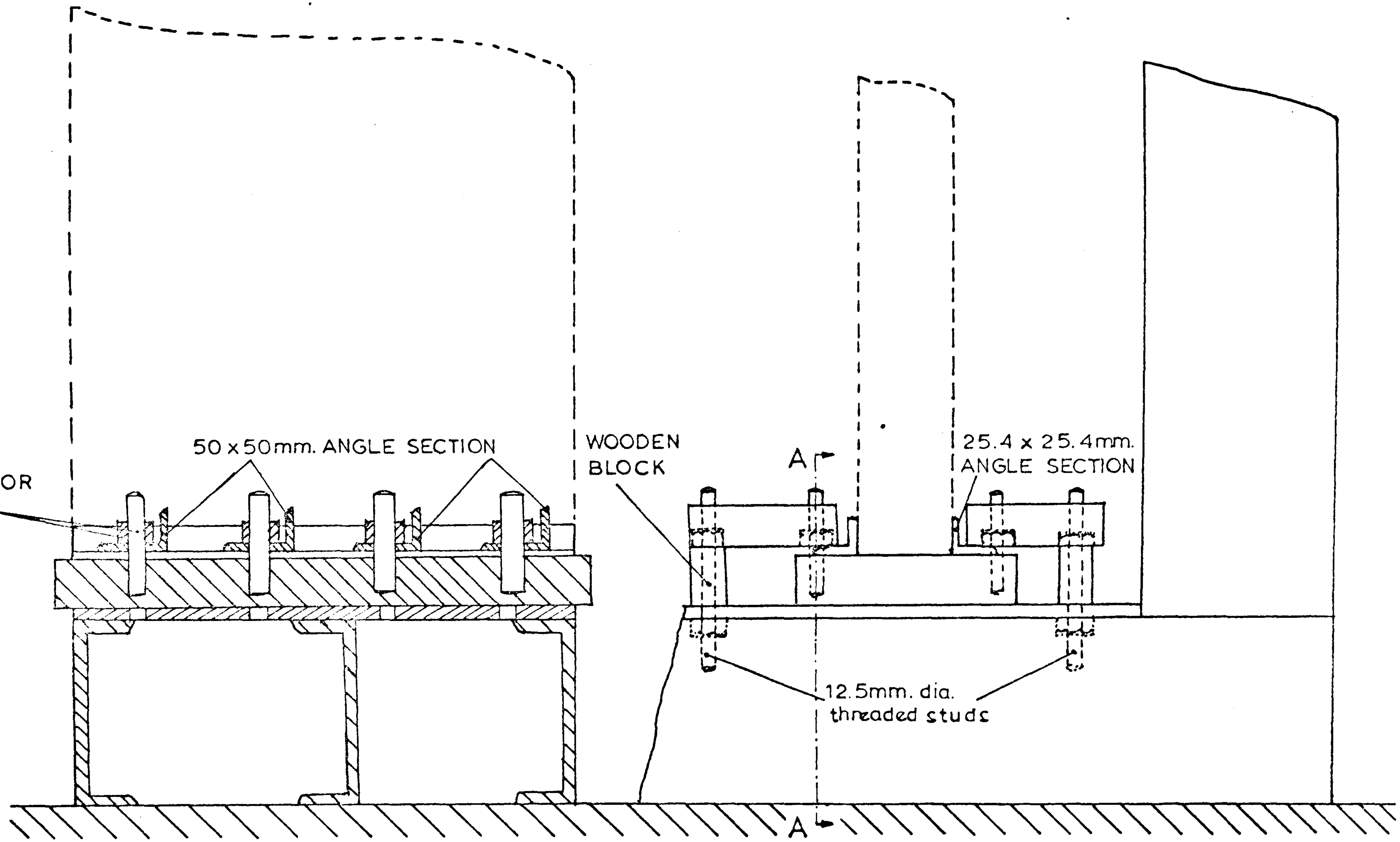
BOLTING OF MODEL 5 ON TEST-RIG



SECTIONAL END ELEVATION
AS IN FIG. 4.12

Fig. 4.14

BOLTING OF MODEL 6 ON TEST-RIG



SECTIONAL END ELEVATION
(A - A)

PART ELEVATION

Fig. 4.15

SINGLE-LINE PULLEY FRAME

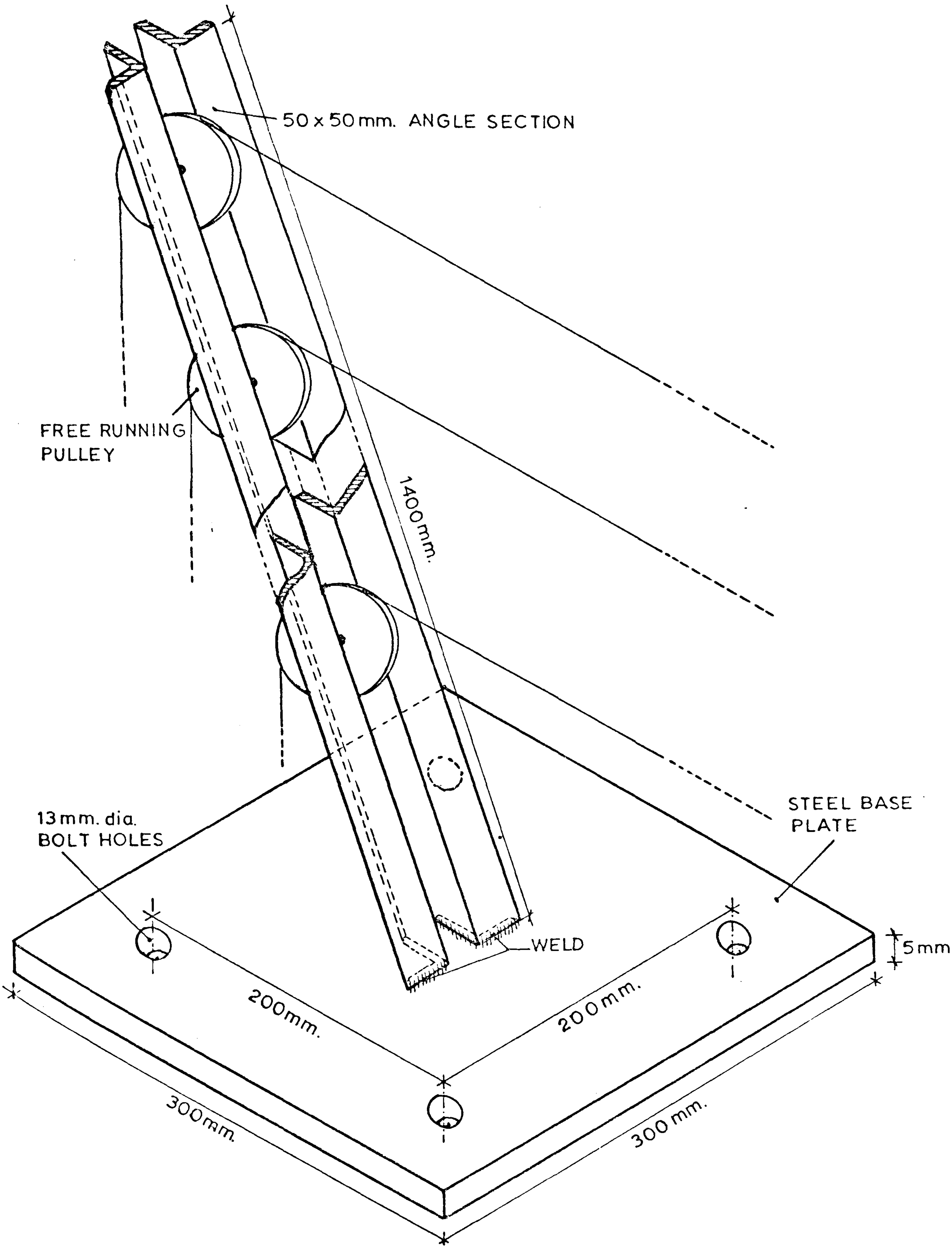


Fig. 4.16

MULTI-PULLEY FRAME

Fig. 4.17

TEST RIG FOR TORSOIN TESTS

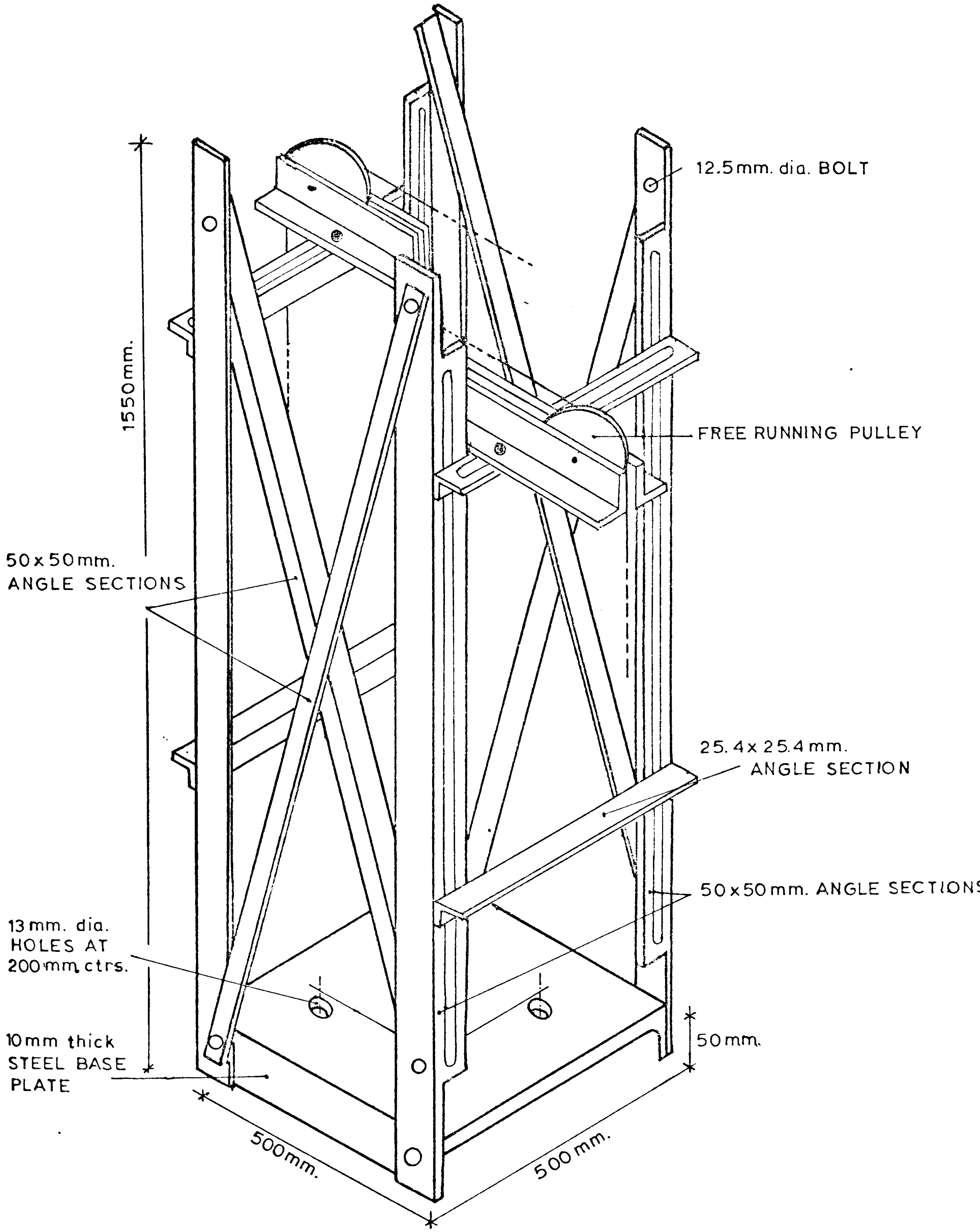


Fig. 4.18

JIG FOR APPLICATION OF TORQUE ON MODEL 2

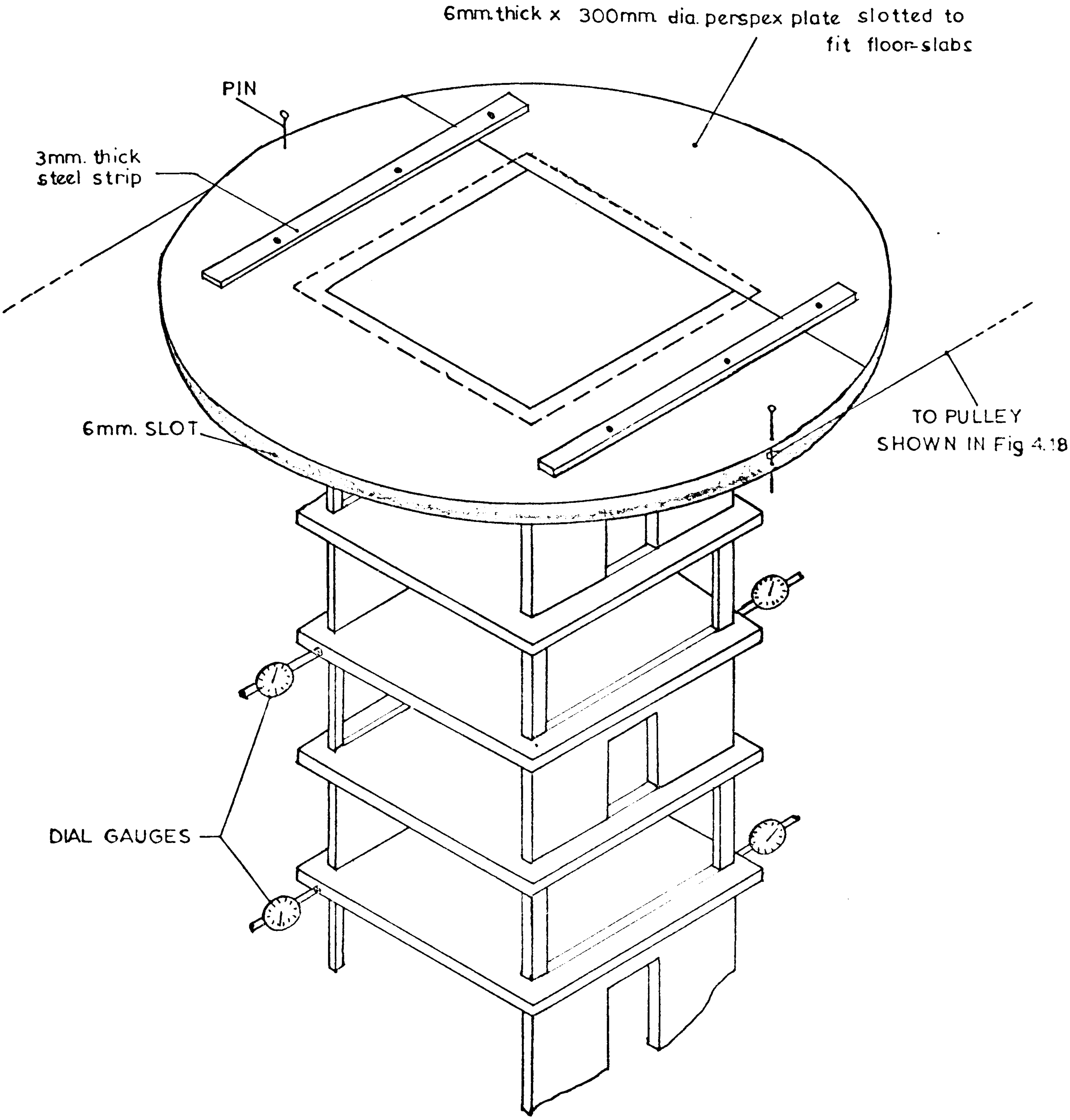


Fig.4.19

CHAPTER 5

5.1 INTRODUCTION

The experimental and theoretical results for each model are presented in the figures and tables of Appendix B. The lateral deflection of the structures and corresponding strain diagrams, (with the exception of Model 3), are given graphically for the load cases considered. Additional curves show the rotations and load distributions of Model 5 and 6, and a comparison between experimental and predicted deflections for load case 1 employing various numbers of reference levels. Deflections and strains of each frame-bent in Models 2, 3 and 4 are given separately.

All loading cases for the models are given in Table B.1 and shown in the appropriate model plans of Appendix A. It should be noted that the model plans relating to the strain diagrams are not to scale and only those strains which have a corresponding experimental value are plotted.

Dynamic response curves, natural frequencies, mode-shapes and estimates of damping present in the structures are given for Models 3, 4, 5 and 6; whilst/

whilst only the fundamental frequency and a measure of damping are given for Models 1 and 2.

The response of each concrete model from a small initial load up to failure load is presented in the form of load/deflection curves. These tests, which were originally intended to be cyclic tests, subjected the models to a centrally positioned point load at the indicated levels.

5.2 THEORETICAL RESULTS

The plane-frame and continuous connection technique described in Chapter 3 were used to compute the deflections, rotations and strains on the Models, with the material properties and model dimensions as given in Appendix A.

Assuming rigid floor-plan movement, the effective width of floor-slabs considered to act as beams coupling walls (78) was included, and local end deformations of the lintel beams (62) were accounted for as indicated. Theoretical estimates of natural frequencies and corresponding mode shapes were determined for Models 5 and 6 employing the method detailed in Ref.(12) and outlined in Section 3.6 The value of Young's Modulus assumed

in/

in calculations was that found using the method described in Appendix D.

5.3 EXPERIMENTAL RESULTS

The experimental values of deflection and strain are superimposed on the theoretical curves. Owing to the relative weakness of concrete in tension the applied loads during the low strain static tests had to be kept small resulting in relatively small experimental readings. Whilst dial gauge readings were generally consistent, electrical resistance strain gauge readings were subject to some scatter despite attempts to insulate the gauges from atmospheric conditions. For every gauge in each test the 'best' straight line was drawn by eye through the plotted results. The final straight-line graph for each gauge was drawn using the mean value of slope from each test in the series. These straight-line graphs were used to determine the strains and deflections for a particular load case. To minimize arithmetical error a small computer program was written for the statistical reduction of the experimental readings.

The dynamic response curves are plotted for the full range of frequencies considered and the relative/

relative amplitudes of deflection at the top of the models. An indication of where the natural frequencies occur is given and more detailed testing in these regions determined more accurate values. Also indicated on these curves are values at which the combined model, actuator and test-rig were found to resonate. Normalized mode-shapes have been presented in which the top-most deflection has been assigned a value of unity with all other deflections adjusted accordingly. This was done to enable a comparison with theoretical values since the energy input to the systems were unknown. Estimates of damping were found using the methods described in Appendix D.

The mode of failure of the micro-concrete models can be seen in the photographs of Appendix E.

CHAPTER 6

6.1 DISCUSSION OF EXPERIMENTAL RESULTS

The test procedures used in the experimental program were designed to minimize the loss of accuracy inherent in any such investigation. Sources of error which do exist and are relevant to this present study are discussed below.

Whilst a high degree of accuracy can be attained in machining perspex models, it is very difficult to achieve a similar degree when casting models in concrete. The expense of perspex or steel shuttering is high and generally timber was used. This limitation combined with the use of polystyrene inevitably lead to variations in the cross-sections of the type of models studied here. These variations can be measured on the final product and accounted for in any subsequent analysis where necessary, such as when using plane-frame techniques. The magnitude of error created by wire-mesh reinforcement drifting slightly off-centre is difficult to assess; but it is likely to be very small since its inclusion was/

was to satisfy minimum code requirements to control temperature and shrinkage cracks.

Generally in model testing, rigid foundations are difficult to achieve. In the case of Model Number 6 lateral movement of the base slab was recorded during tests. The extent of this was approximately 2×10^{-2}mm over the entire load range during static tests. The industrial grease used to position the model on the test-rig probably contributed to this and despite attempts to alleviate the problem by bolting angle-sections on the test-rig and against the base slab as in Fig. 6.1, some movement still occurred. A load/deflection graph for this movement was drawn and the experimental results adjusted accordingly. No rotations of any of the model bases were recorded but deformations within the bases themselves may have occurred and influenced the results. No significant 'out-of-plumb' between the vertical structural elements and horizontal base slabs existed.

Loading on the models was applied directly by a system of dead weights, as described in Chapter 4. The/

The perspex models required much smaller loads than those available and therefore consisted of small polythene sacs containing 25gm of Leighton Buzzard Sand. These were numbered and weighed on an accurately calibrated electronic balance. A total of 100 weights were made with a mean value of 24.98gm and standard deviation of 1.2×10^{-2}gm. Any error arising from an incorrect magnitude of load would therefore be very small.

It was generally found that the dial gauges required several load increments before settling in and thereafter they behaved uniformly producing consistent results over a number of tests. The small readings associated with the concrete models produced a set of results through which a straight line could be regarded as a good fit. The larger deflections produced in the perspex model tests exhibited linearity. During the tests in which Model Number 2 was subjected to a torque, the tips of the gauges tended to slip on the perspex surface. This problem was overcome by applying a small amount of 'Durofast' contact adhesive to both surfaces; this could easily be removed after testing.

The/

The presence of creep and micro-cracking in the model materials will result in a loss of accuracy. This loss can be minimized by adhering to a strict time schedule for perspex models, and low strain tests for concrete models.

There is no readily available method of assessing the accuracy of the strain gauges and recording equipment. On Model Number 3 the strain gauges proved unreliable. This may have been due to surface irregularities, short-circuiting of the leading wires between the gauges and terminal strips caused by surface moisture or drift due to differential heating between the dummy and active gauges. Additional precautions such as isolating the leading wires from the concrete surface and using two dummy gauges alternatively, did result in useful readings on later models. The laboratory conditions were such that there was little control over atmospheric conditions. Wooden partitioning and polythene sheeting were erected around the immediate vicinity of the test-rig in an attempt to isolate the area from draughts and local temperature changes.

Assessing/

Assessing the overall accuracy of experimental results is very difficult since they can be influenced by many factors; some of which are cumulative, others which are compensating.
The deflections and rotations can be expected to be within $\pm$ 5% to $\pm$ 10% of the true values.
Strains due to bending and direct stresses are likely to be in the region of $\pm$ 10% for perspex models where the material parameters and model dimensions are reasonably consistent, those on the micro-concrete models are probably within $\pm$ 25%, but this could be higher with the small readings recorded during low strain tests.

The dynamic response of the structures were measured directly using an accurately calibrated oscilloscope with a store facility. This enabled an accurate evaluation of the response curve and therefore little error can be expected here.
At the lower end of the frequency range (0 - 34Hz) both dynamic strain records and accelerometer output were used to select natural frequencies.
This was necessary since the energy input in this range was mostly dissipated in displacement with consequent high amplitude vibration, which had to be reduced to minimize the risk of damage to the models./

models. The fundamental period, can be checked using 'pluck test' data (e.g. response curve on an ultra-violet recording). Any cracking present in a concrete model will produce a more flexible system with a natural frequency which is lower than the true value (30).

6.2 DISCUSSION OF THEORETICAL RESULTS

The assumption of a linearly elastic, homogeneous, continuous and isotropic material in a structure is the major source of variation between predicted and real behaviour.

Material parameters such as Modulus of Elasticity and Poisson's Ratio, which are found experimentally and consequently are approximate values, have a direct influence on the predicted static and dynamic behaviour. The values of these parameters are probably the major source of error, particularly when dealing with a material such as micro-concrete.

In general, it is unlikely that an analytical model of a shear-wall structure will be seriously in error by assuming the floors fully rigid in their own plane.

The/

The theoretical estimates could be improved by including the additional torsional stiffness of the staggered-frame-bents and allowing for coupling between the plane-units in torsion, additional warping stiffness of units and elastic deformations of the wall-base intersections. In view of the assumptions mentioned in the first paragraph, such refinements may not be justified.

The continuous connection technique assumes the topmost connecting beam of the building to be one-half the stiffness of the other beams. In Models 5 and 6, all beams were equal, but since they were not considered to be of sufficient stiffness to prevent rotation of the end of the walls, this assumption will lead to little error.

6.3 COMPARISON BETWEEN THEORETICAL AND EXPERIMENTAL RESULTS

Deflections

In the perspex models, which come close to having ideal elastic properties (if testing is carried out to minimize temperature, humidity and creep effects), a close agreement is reached between experimental and theoretical deflections. Several papers have shown/

shown that a frame analysis can give adequate predictions of deformations in regular wall systems (56, 57, 78) and the results of this present study emphasize the suitability of the frame-element mentioned in Section 3.2 in analysing staggered frame-bents.

The differences between theory and experiment in Models 2, 3 and 4 are increased due to the approximate load distribution assumed between the two frame-bents. These models are asymmetrical, single-bay structures and cannot therefore, be expected to behave in the same manner as two adjacent frame-bents within a much larger structure. The frame-bents are not restrained by adjacent structural units and floor-slabs and therefore do not necessarily deflect by the same amount (as assumed by Derecho (21) for two typical frame-bents of a structure) when subjected to a centrally positioned lateral load. This asymmetry is reflected in the different magnitudes of the deflections of the separate frame-bents. A 50% load distribution gives more accurate predictions for Model 2 and 3 but this value is dependent on the relative stiffness of the columns and wall-beams at any one level/

level, which can be better approximated using the method suggested in Ref.(21).

Wider discrepencies can be expected in the results from the concrete models because of the presence of micro-cracking, the assumption in the theory of a material with a linear stress/strain curve and the less accurate determination of modulus of elasticity.

In Model 4 very large differences are shown between theoretical and experimental deflections. This can be attributed to two factors in addition to cracking:

1) during construction of this model remedial repair work was required on the floor slabs. In some cases this was considerable, (Plate 3), thus effectively decreasing the stiffness of the system. The floor-slabs transfer horizontal shear-forces which can be high particularly at lower levels, and consequently the more flexible system results in greater deflections than would otherwise be expected.
2) The value of modulus of elasticity was found from the results of a crushing test on a cylinder which when compared to other values, is slightly high and/

and must be treated with suspicion. These factors have obviously had a marked effect on the measured and predicted deflections and a constructive comparison between theory and experiment is impossible.

In the case of all Models, a uniformly distributed load was simulated by applying loads at intermediate levels and not at every level as used for the theoretical analyses. This will lead to a slight difference between the predicted and measured deflections.

Experimental and theoretical values of deflections and rotations for two positions of a point lateral load and uniformly distributed lateral load on Models 5 and 6 are given in Figs. B-5.1 to B-6.16. Generally good agreement is reached, those of Model 5 being better than those of Model 6.
The larger discrepencies in the results from Model 6 can be explained by the base movement mentioned previously and possibly a more flexible junction between the structure and base slab where additional concreting was required after casting.

The/

The rotations obtained from the theoretical analyses are greater than those found by experiment. In the analyses the torsional stiffness of plane shear-walls and coupled channels were accounted for; the warping stiffness of all elements except the cores, the coupling effect of shear-walls in torsion and the torsional stiffness of frame-bents were ignored. These omissions result in an underestimate of the torsional stiffness of the structures.

The approximate analyses of Model 5 using the simplified shear-wall structure shows differences in the region of 5 - 7% for deflections and 3 - 4% for rotations from those obtained using a combined shear-wall and plane-frame approach. A great deal more research is required to establish the limitations of this approximate method since only one model of a regular system has been tested. The accuracy of the solution may be dependent on several parameters such as "width of wall-beam/storey height" ratio, the "width of wall-beam/width of opening" ratio or the in-plane stiffness of the floor-slabs. One of the advantages of the continuous connection technique is/

is that a reasonably accurate solution can sometimes be found employing fewer reference levels, than actually exists. Curves of the relative accuracy have therefore been drawn for the approximate values of maximum deflection using the simplified shear-wall structure of Model 5. The values obtained using the chosen numbers of reference levels have been compared with those found using the maximum number of fifteen levels employing the shear-wall/frame analysis. This was used as a basis for comparison since it is a more accurate representation of the system and compares favourably with experimental results.

The indications from Fig. B-5.1 to B-5.16 are that the complexity of a structure such as Model 5 can be reduced using the simplified method of analysis with slightly fewer numbers of reference levels, to obtain a rapid assessment of structural behaviour.

Strains/

Strains

Theoretical and experimental strains at the test sections, are given for all models except Model 3.

A favourable comparison is found for the strains measured on the walls and columns of the perspex models, particularly Model 2. From an examination of the strain diagrams of Model 1, it can be seen that the cclumns are principally axially loaded members, with bending strains in the region of only 6% of axial strains. In the theoretical analysis, this pattern followed throughout the entire height of the building illustrating the deflected form of the frame as a cantilever bending in single curvature.

As with deflections, the strain measurements on the micro-concrete models show longer discrepencies from the predicted values. In many cases the values are very small and can be attributed to the 'drift' phenomenon in electrical resistance gauges. Generally there is a good agreement with the theoretical values of Model 5 where the gauge readings are of sufficient magnitude to discount drift.

Load Distribution/

Load Distribution

In some simple design procedures the lateral loads on shear-wall buildings are sometimes distributed among the vertical load bearing elements according to their cross-sectional properties. Such methods can lead to gross errors when all walls are not identical and large interchanges of load can occur between the different elements throughout the height of the building. It is possible to obtain complete reversals of load as shown in Figs. B-5.6, B-5.15, B-6.5 and B-6.12. These figures highlight the necessity for a complete three-dimensional analysis of complex structures to obtain reliable design quantities. In addition to this Figs. B-5.6, and B-5.15, illustrate the 'flow' of horizontal shear throughout the staggered-frame-bents.

Dynamic Response

Only experimental results are given for the dynamic tests on Models 3 and 4 (and the torsion tests of Model 2). The response from Model 4 appears very erratic and like the deflection and strain diagrams are included only for completeness of the test program results.

From/

From Figs. B-5.21, B-6.14 and Tables B-5 and B-6, the indications are that the theory presented in Ref. (12) predicts estimates of reasonable accuracy of the first, and to a lesser degree, the second natural frequencies of the systems studied here. Only three co-ordinates are plotted for each set of results in Figs. B-21 and B-26 and consequently only a general indication of any relationship between natural frequency and mode is possible. The first and second mode shapes show a reasonable comparison with experimental results, but the third is widely different.

Considering the lack of information on the dynamic response of tall complex structures, these results are encouraging and illustrate the value of approximate methods of analysis such as that presented in Ref. (12).

Failure Modes of Micro-Concrete Models

Model 3: The load deflection curves show a marked increase in the deflections of frame-bent A at 42-47N. Despite this increase in torsional deformation, no visible cracking appeared until a load of 196N. At this load a small crack appeared at ground level on the tension column of frame-bent A. Two further/

further load increments resulted in very gradual creep as at all other load increments but continuing, until after approximately 1.5min. had elapsed, then failure occurred. During this period of 1.5 min diagonal cracking appeared in the first floor slab which failed due to torsion, this was rapidly followed by the failure of the columns on level one of frame-bent B.

Model 4: As with Model 3, the asymmetry of this structure was emphasized by the torsional deflection and mode of failure. Cracking first appeared at 117.5N, on the tension column at ground level, a further load increment initiated diagonal cracking and subsequent failure of the first floor-slab as in Model 3. The ultimate load capacity of this structure was obviously reduced by the weakened floor-slabs.

Models 5 and 6: The load/deflection curves for the ultimate load tests on Models 5 and 6 are given in Figs. B-5.22 and B-6.17, respectively. The graphs indicate flexural failure with no signs of torsional deformation.

Cracking/

Cracking first appeared on Model 5 at 1500N, 50mm above the base of unit H, and on Model 6 at a load of 1800N at the base of units F, D and C. One further load increment of 2kg on Model 5 caused flexural cracking on all other units at base level and subsequent failure at a load of 1525N. The mode of failure was slow continuous deformation for about 10 seconds, then collapse.

Several load increments of 5kg were applied to Model 6 before the crack pattern fully developed on every unit at a load of 2112N, when failure occurred. The mode of failure was the same as in Model 5; flexural cracking with gradual creep until collapse.

After the ultimate load tests both models were undamaged except for the first storey and could possibly be suitable for re-casting into another concrete base and further testing.

6.4 CONCLUSIONS

1) Existing theoretical analysis techniques for high-rise shear-wall structures are applied to two-dimensional and complex three-dimensional staggered and shear-wall tall building systems. The analyses presented make use of the continuous connection method in assessing the contribution of uniform, vertical load bearing units to the overall structural stiffness and simultaneously utilize the plane-frame approach in assessing the contribution of staggered-frame-bents. The analyses of the three-dimensional systems are suitable for any loading which may cause bending and torsion of the structure.

2) A method of rapidly obtaining an approximate solution and preliminary assessment of the structural behaviour of a non-uniform staggered-wall-beam system is illustrated using the continuous connection technique. The limitations of the method have not been determined.

3) Comparison of the results from dynamic testing and the theoretical analyses have indicated the suitability of the approximate dynamic analysis presented/

presented in Ref. (12) for estimating the first few natural frequencies of regular symmetric shear-wall structures.

4) Methods suitable for the fabrication of complex small-scale reinforced concrete structures are presented. The methods are illustrated with the construction of two single-bay, staggered-wall-beam structures and two five-bay, fifteen storey staggered and shear-wall building systems.

Providing sufficient attention is given to the size of the constituent materials of the concrete, fixity of the polystyrene moulds and vibration techniques, models of a high standard can be produced.

These models, in addition to a staggered-wall-beam plane-frame and a three-dimensional single-bay S.W.B. structure which were fabricated from perspex, were subjected to static and dynamic tests. The results of the static tests produced values of deflection, rotation and strain, whilst those of the dynamic tests gave natural frequencies, mode-shapes and a measure of the damping present in the structures.

5) An analytical and experimental correlation is given of the results from static and dynamic tests on the models and those from the theoretical analyses. This indicates the suitability of the analyses for both preliminary and final design purposes. Experimental results only, are presented for ultimate load tests on the micro-concrete models.

6) The ultimate load capacity and load/deflection graphs of Models 5 and 6 indicate that selective replacement of several consecutive coupled-shear-walls in a cross-wall structure by staggered-frame-bents, can result in a structural system which still possesses a high percentage of the original stiffness and strength. This alternative system provides much greater flexibility and latitude for architectural planning and expression.

7) The performance of shear-wall buildings during earth tremors has indicated their ability to withstand large seismic loadings. The similarity in the dynamic response of the shear-wall and S.W.B. models found here, together with cyclic tests to determine ductility characteristics, may provide evidence to support the view that S.W.B. systems can also respond favourably to severe earthquake forces.

8) The necessity for three-dimensional analyses of complex high-rise buildings is shown analytically by the lateral load distribution curves of Models 5 and 6.

6.5 SUGGESTIONS FOR FUTURE RESEARCH

6.5.1 EXPERIMENTAL

Very little work, particularly model studies, has been done on the dynamic and elasto-plastic characteristics of complete high-rise buildings. The methods presented for the fabrication of small scale concrete buildings should enable a wider series of tests to be conducted on existing and recently developed structural systems. Extensive experimental work should be carried out to provide data for comparison with analytical analysis techniques. A range of problems which can be investigated are given below:

a) Static and dynamic response of buildings, varying parameters such as plan-layout, reinforcement arrangements, and footing conditions.

b) Ductility characteristics of structures and individual structural elements by cyclic loading at near failure.

c)/

c) The response of structures at near failure conditions when vibrating at a pre-determined natural frequency or subjected to an earthquake input record to yield information regarding structural fatigue and potential structural damage.

d) Collapse modes of proposed structural systems.

e) The effects of explosions, structural modifications and new details such as damping systems on structural response.

f) Assessment of proposed demolition techniques.

g) Response to irregular load patterns.

h) Detailed testing of particular building sections to yield information regarding the mechanics of load transfer between structural elements.

Experimental data from tests such as those described above can be invaluable in defining the limitations of existing theoretical analyses.

In the present study if, in addition to reducing the bending moment at the base of Model 6 by applying the load at level 11, much smaller load increments had/

had been available, cyclic load tests may have been successful. An amplifier connected between the accelerometer and recording equipment during dynamic testing would have improved the accuracy of the resulting mode shapes.

6.5.2 THEORETICAL

a) The development of dynamic and ultimate load methods of analysis is incomplete and at present most techniques are only available in research papers (12, 22, 27, 47). In view of the tendency to construct high-rise buildings in earthquake prone areas there seems an urgent need for rigorous analysis techniques being readily available to design engineers.

b) The torsional stiffness of structures is often underestimated by ignoring the warping stiffness of most structural elements and the stiffness due to coupling of shear-walls in torsion. Studies to determine the magnitudes of these parameters would lead to more accurate predictions of deflection and rotation, although this is slightly compensated due to neglect of base and joint flexibility.

c)/

c) The approximate analysis described in Section 3.4 showed a favourable comparison with the more lengthy shear-wall/plane-frame approach. Extensive experimental and theoretical investigations are required to determine the limitations of the method.

Additional measure to stop slipping of Model 6

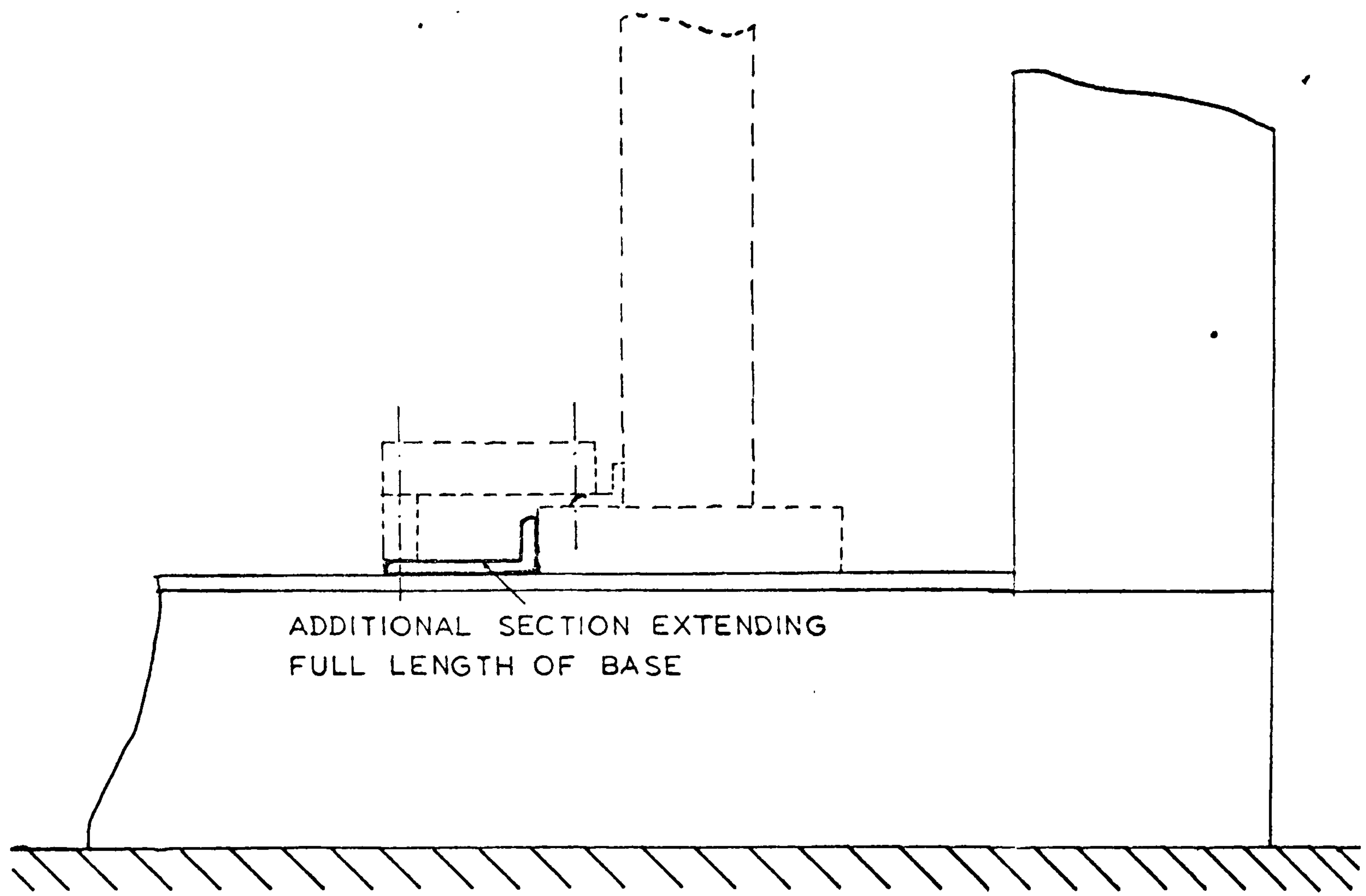


Fig.6.1

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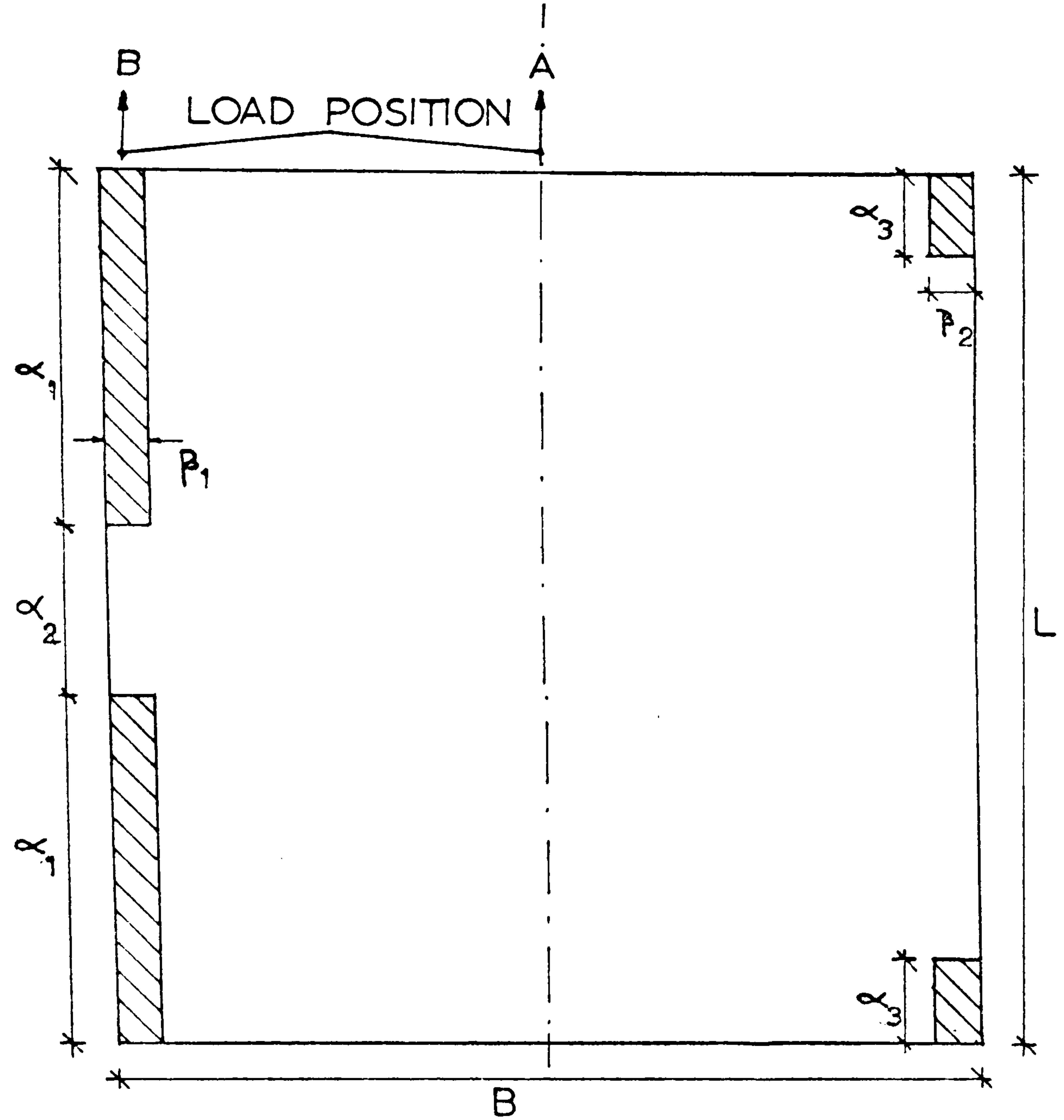
APPENDIX A

Model Dimensions

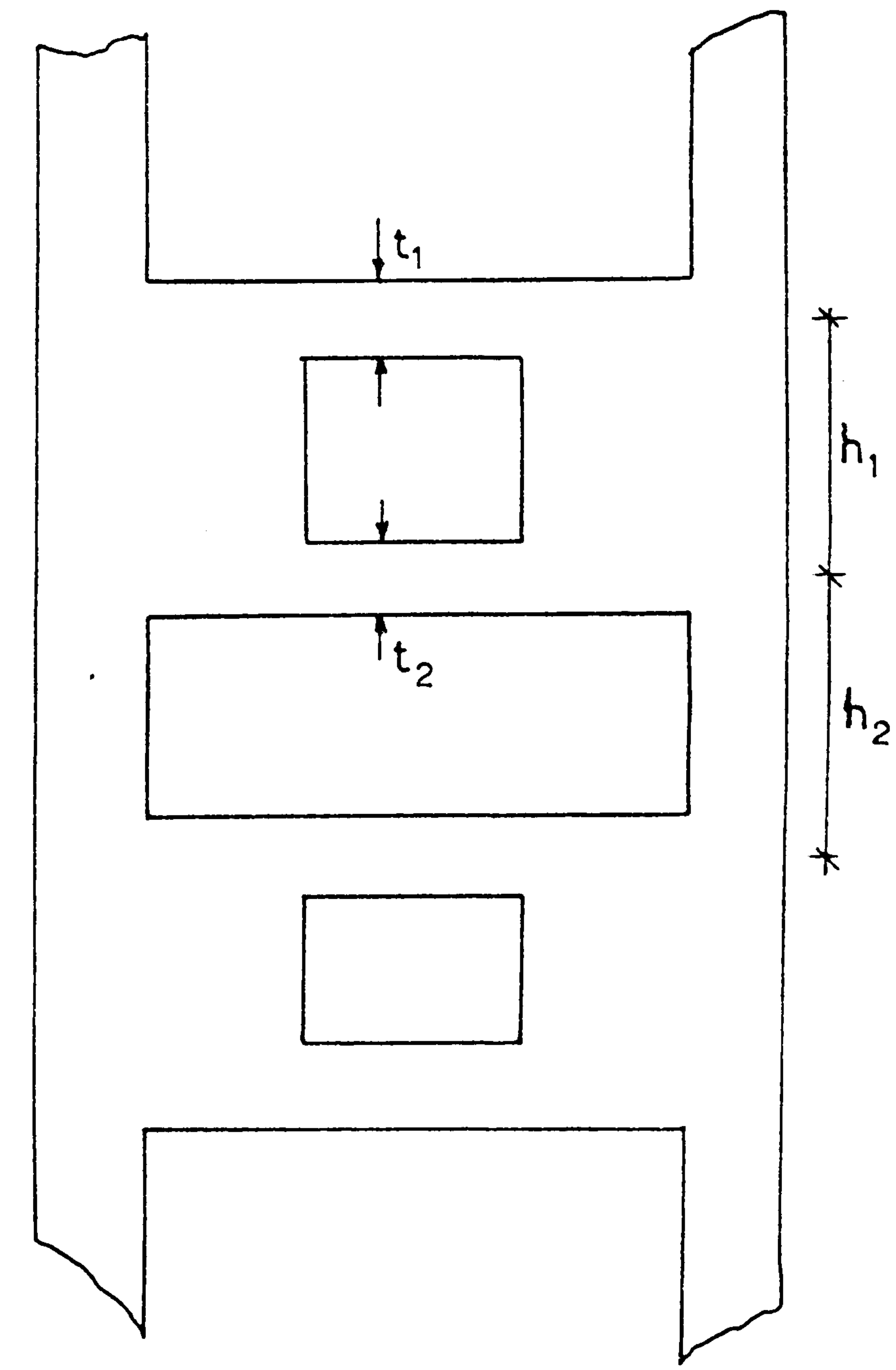
and

Material Parameters



TYPICAL FLOOR PLAN MODELS (1),2,3,4

(See table A.1 for dimensions)



ELEVATION

Fig. A-1

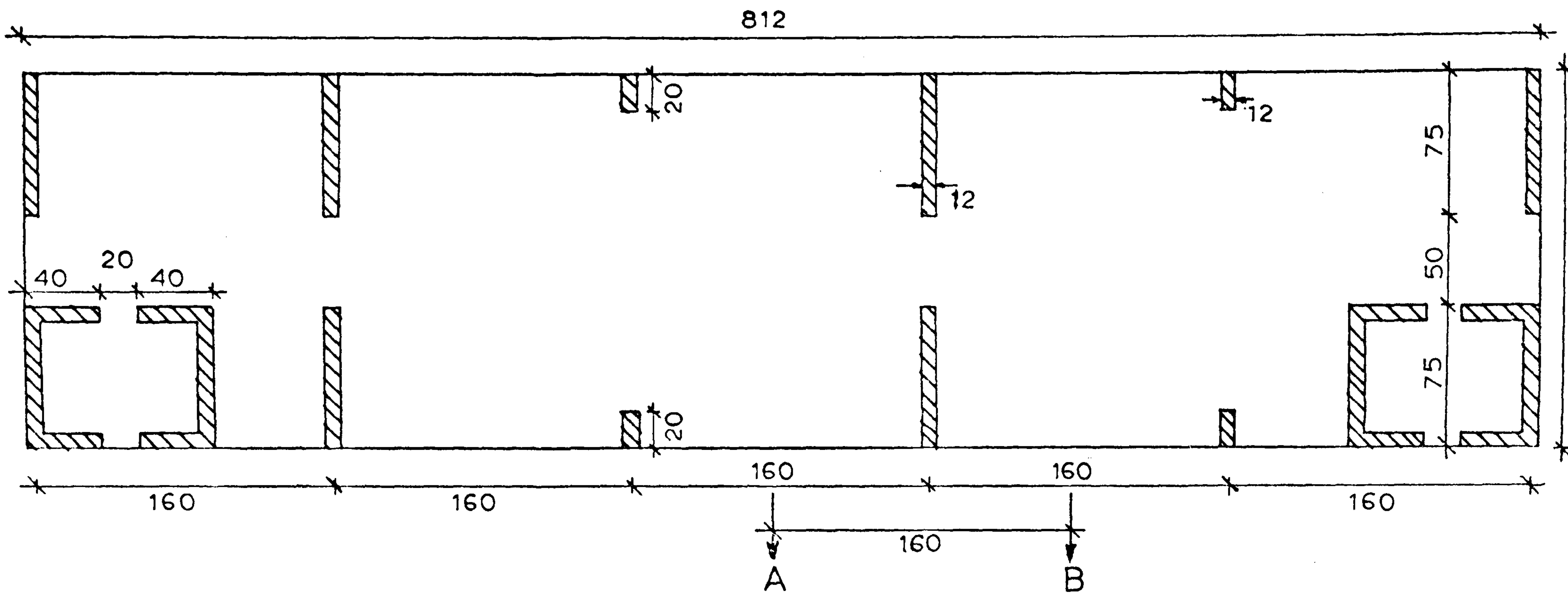
ν POISSON'S RATIO

E YOUNG'S MODULUS (kn./mm.^{2})

All dimensions in mm.

MODEL NUMBER	1	2	3	4
L	160	101.6	300	300
B	0	92.08	400	325
α_1	65	38.1	111.9	110
α_2	30	38.1	76.2	80
α_3	20	6.35	23	25
β_1	12.5	3.175	12.5	12.5
β_2	12.5	3.175	12.5	12.5
t_1	15	9.525	12.5	12.5
t_2	10	9.525	12.5	12.5
h_1	47.5	60.235	89.2	162.5
h_2	52.5	60.235	89.2	162.5
E_{static}	2.790	3.305	32.29	45,50
$E_{dynamic}$	3.61	4.04	43.63	—
ν	0.351	0.34	0.14	0.14

Table A.1



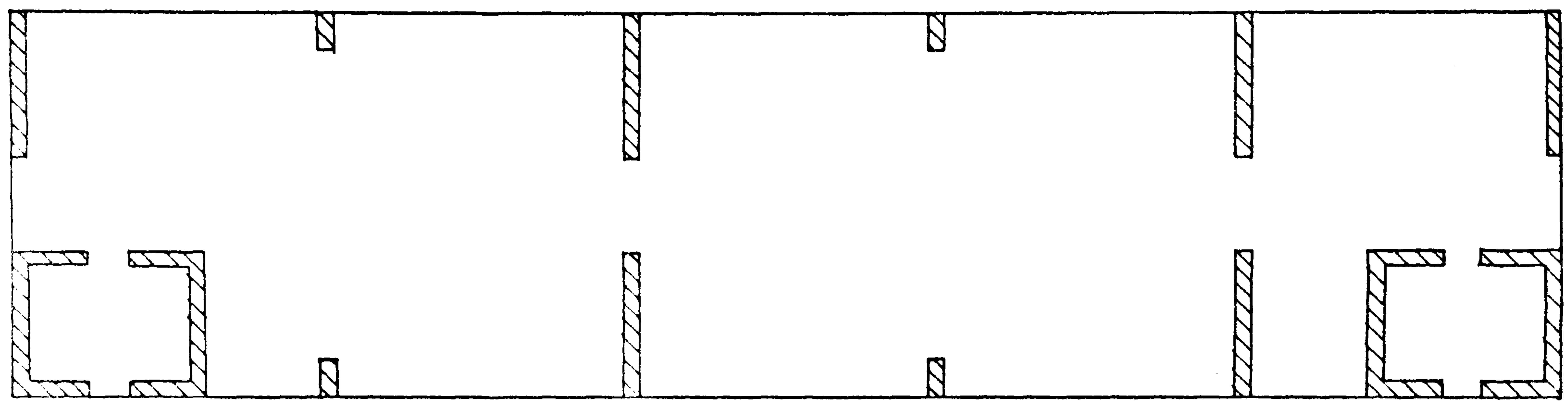
TYPICAL FLOOR-PLAN TYPE A

MODEL 5

A & B – Load Positions

All dimensions in mm.

Fig. A-2.1

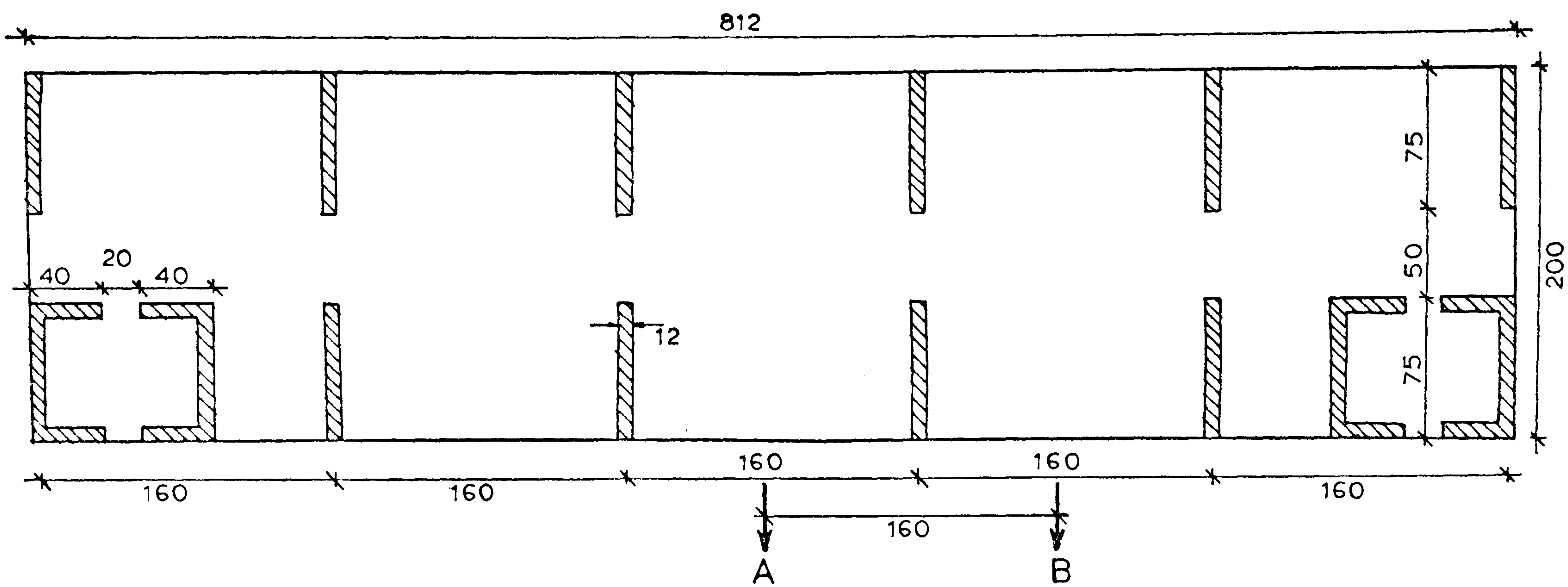


TYPICAL FLOOR-PLAN TYPE B

MODEL 5

Dimensions as in fig A-5.1

Fig. A-2.2



FLOOR-PLAN MODEL 6

A & B - Load Positions

All dimensions in mm.

Fig. A-3.1

MODELS NUMBER 5 and 6	
Number of Storeys	15
Storey Height	125 mm.
E_{static}	27.97 KN/mm^2
$E_{dynamic}$	33.84. KN/mm^2
Poisson's Ratio	0.14
Density	4.073×10 Kg/M^3

Table. A.2

APPENDIX B

Experimental and Theoretical Results

LOAD CASES				
MODEL No.	LOAD CASE	POSITION OF LOAD	LEVELS AT WHICH APPLIED	VALUE OF LOAD (N.)
1	1	B	20	19.62
	2	B	2,4,6 ····20	2.0
2	1	A	20	4.905
	2	A	2,4,6 ····20	0.491
3	1	A	12	58.86
	2	A	3,6,9,12	19.62
4	1	A	11	39.24
	2	A	1,3,5 ····11	14.72
5	1	A	15	196.2
	2	A	1,3,5 ····15	39.24
	3	B	15	196.2
	4	B	1,3,5 ····15	39.24
6	1	A	15	196.2
	2	A	1,3,5 ····15	39.24
	3	B	15	196.2
	4	B	1,3,5 ····15	39.24

Table B.1

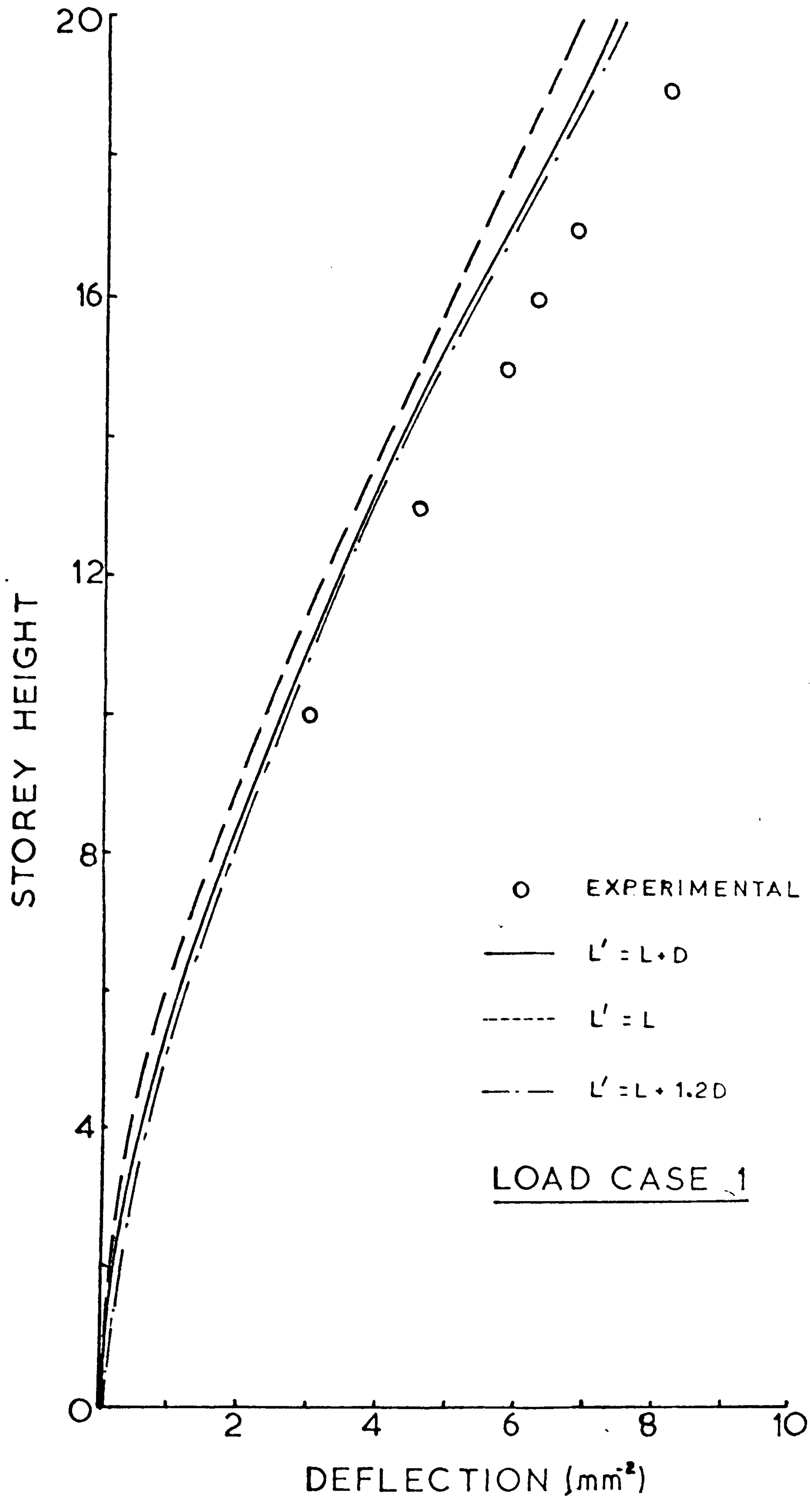
A & B refer to positions shown in APPENDIX A

In the following diagrams

L — Actual length of connecting beams

L' — Effective " " "

Model No.1



20
16
12
8
4
0
STOREY HEIGHT
2
4
6
8
10
DEFLECTION (mm)
EXPERIMENTAL
L' = L + D
L' = L
L' = L + 1.2D
LOAD CASE 1

Fig B-1.1

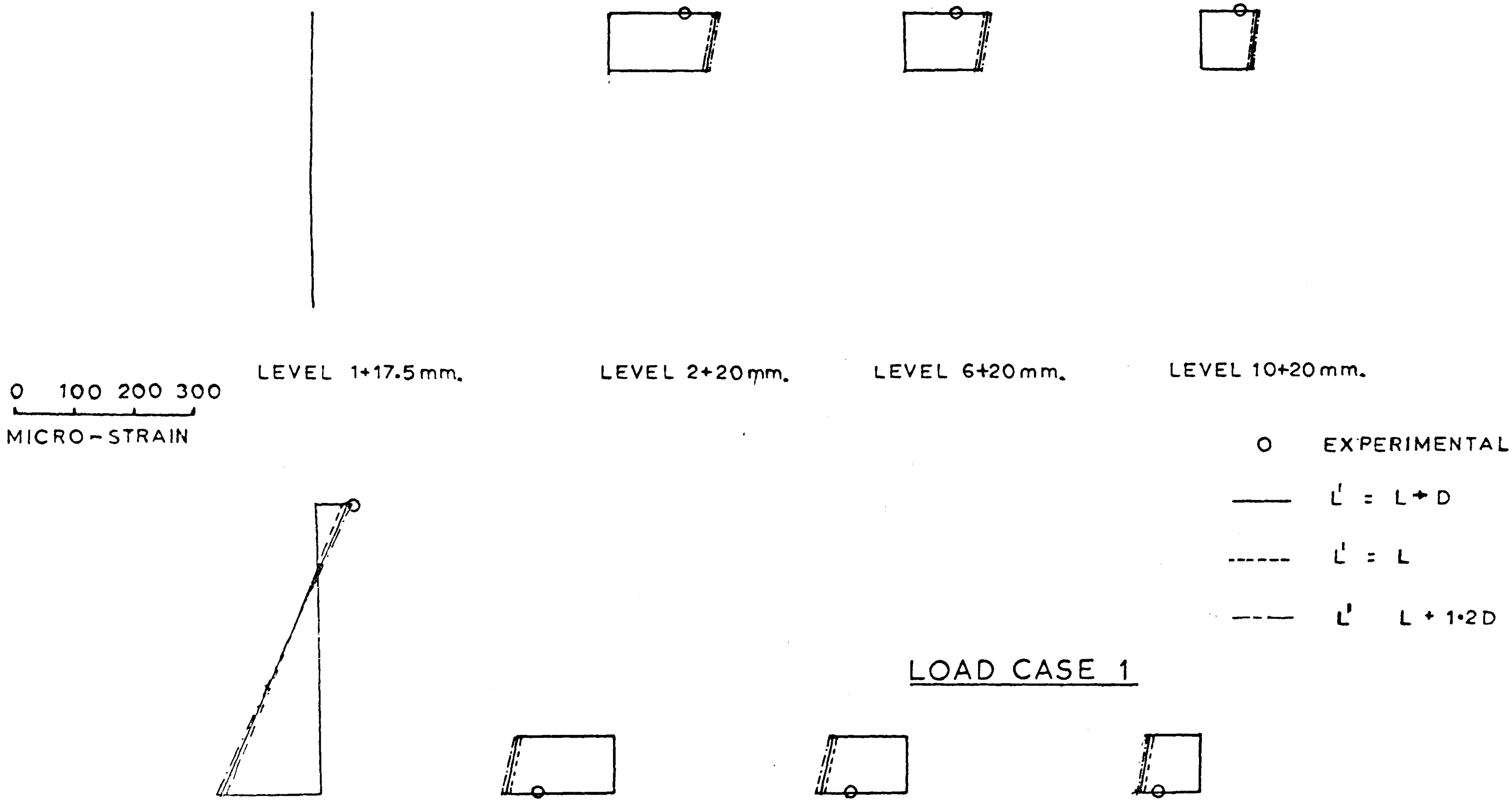
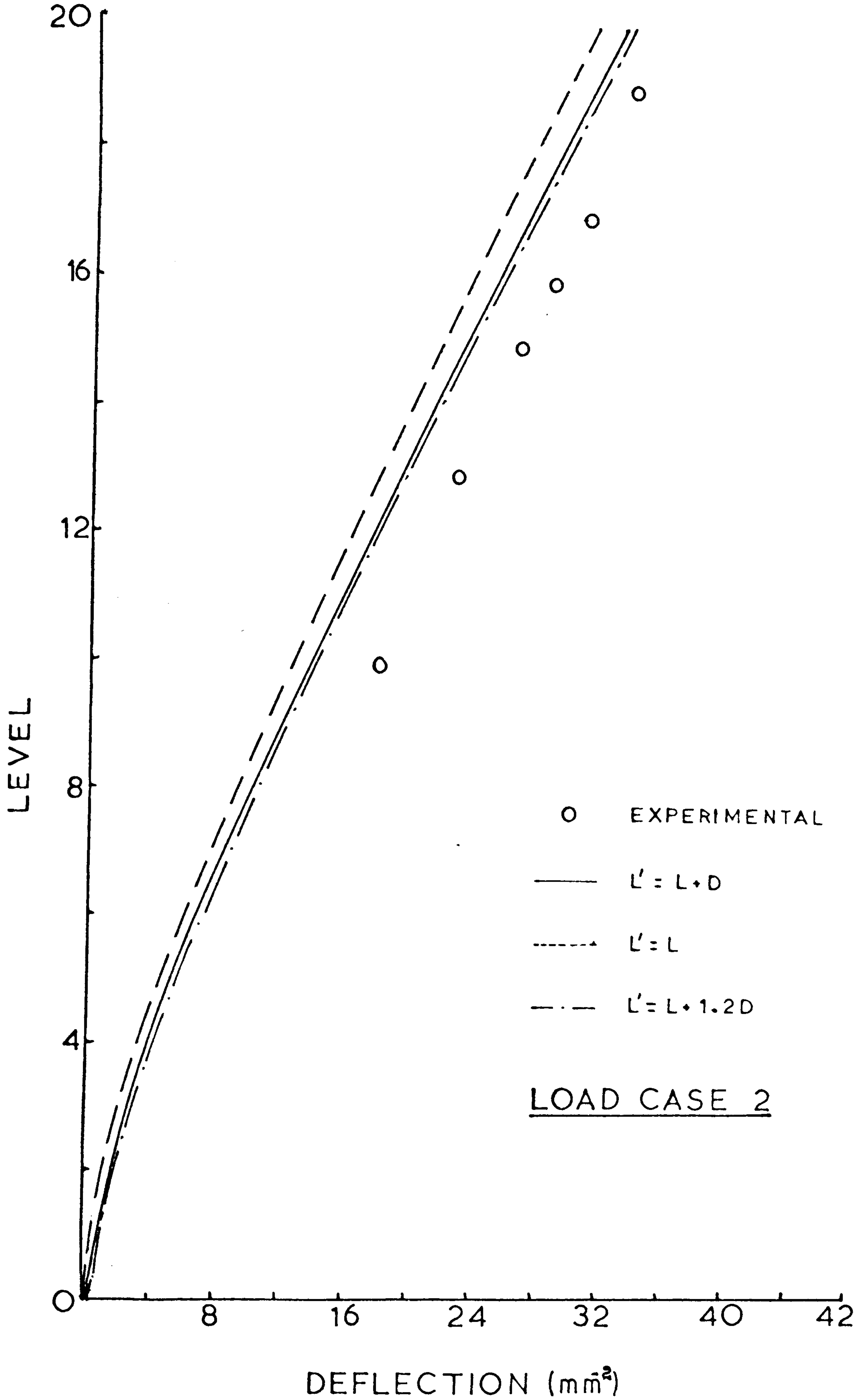

LEVEL 1+17.5mm.
LEVEL 2+20mm.
LEVEL 6+20mm.
LEVEL 10+20mm.
0 100 200 300
MICRO-STRAIN
O EXPERIMENTAL
L' = L+D
L' = L
L' L + 1·2D
LOAD CASE 1

Fig B-1.2



20
16
12
8
4
0
LEVEL
8
16
24
32
40
42
DEFLECTION (mm⁻²)
EXPERIMENTAL
L' = L+D
L' = L
L' = L+1.2D
LOAD CASE 2

Fig B-1.3

LEVEL 1+17.5mm.
LEVEL 2+20mm.
LEVEL 6+20mlm.
LEVEL 10+20
0 40 80 120
MICRO- STRAIN
EXPERIMENTAL
L' = L + D
L' = L
L' = L + 1.2D
LOAD CASE 2

Fig B-1.4

MODEL No. 1

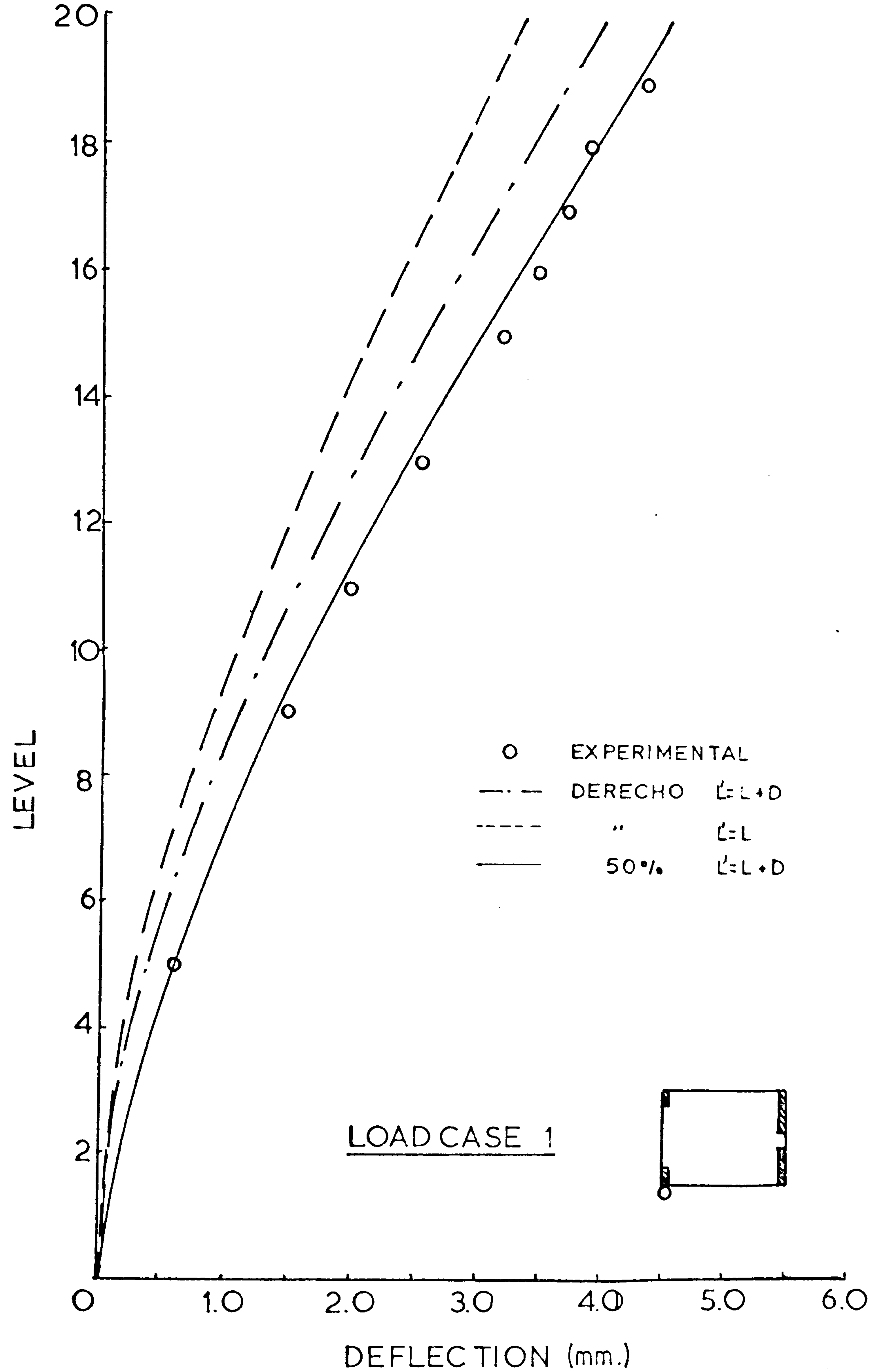
EXPERIMENTAL FUNDAMENTAL FREQUENCY

45.45 Hz

AVERAGE VALUE OF DAMPING

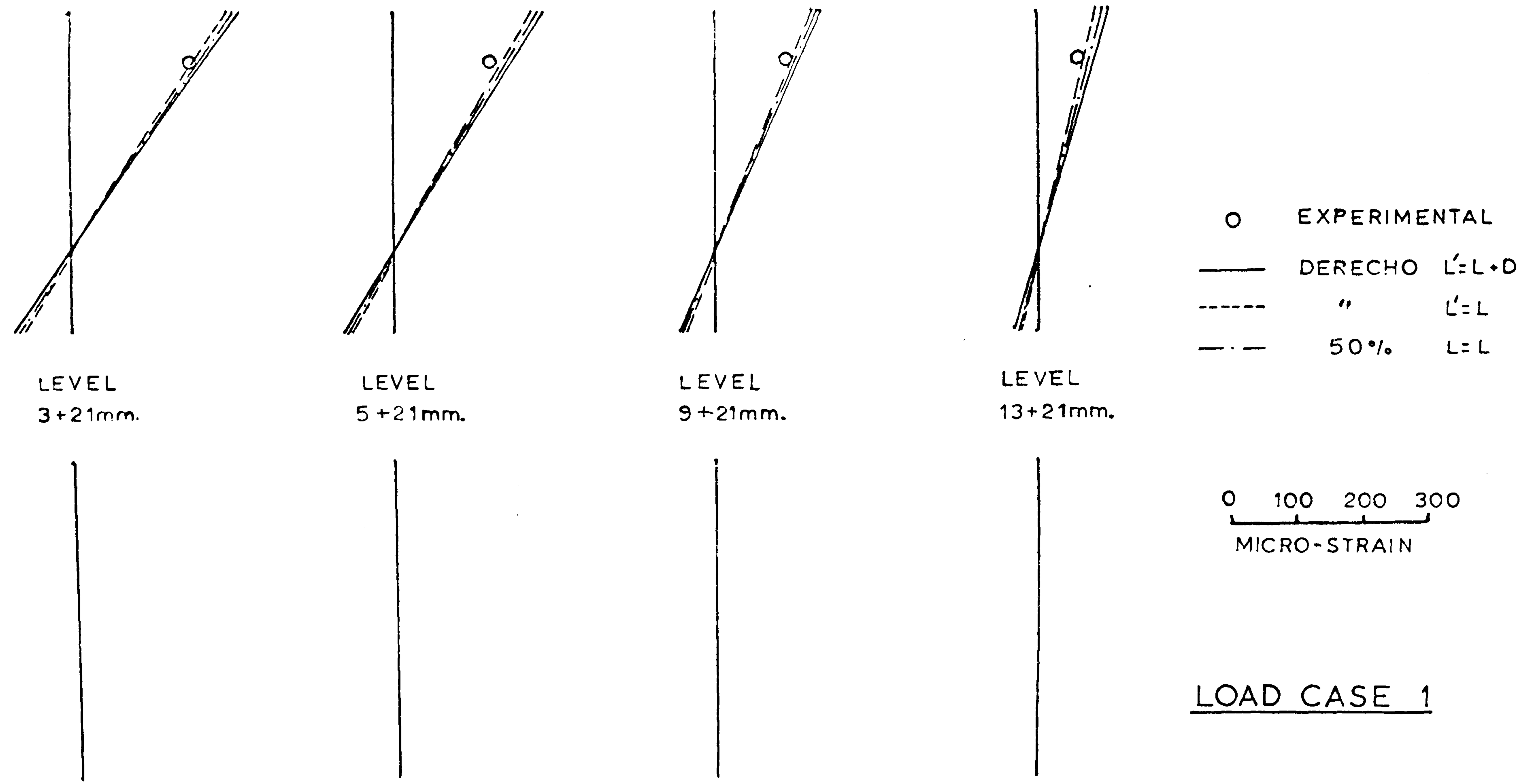
8.34% Critical

Model No. 2



FRAME BENT A

Fig. B-2.1



FRAME BENT A

Fig B-2.2

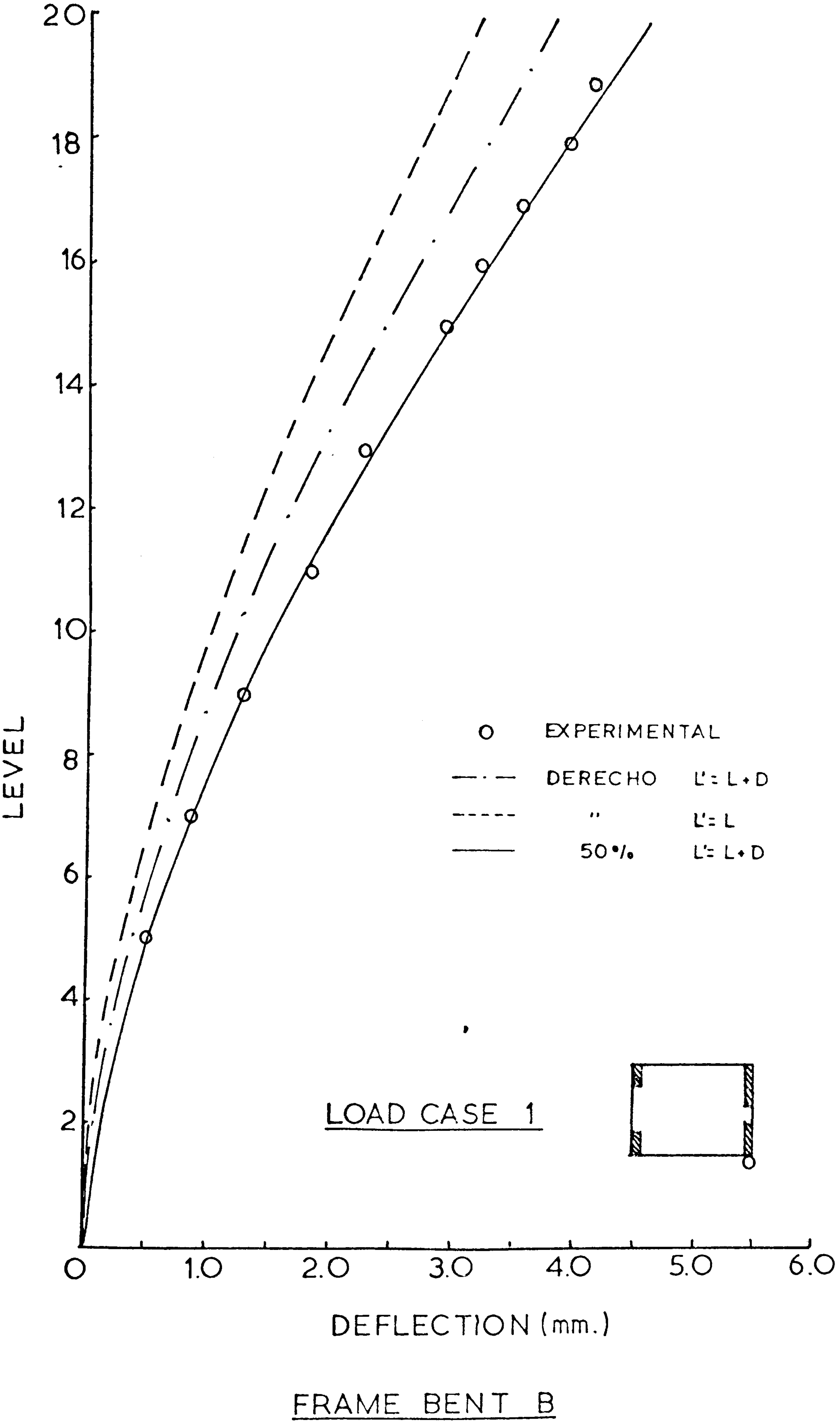
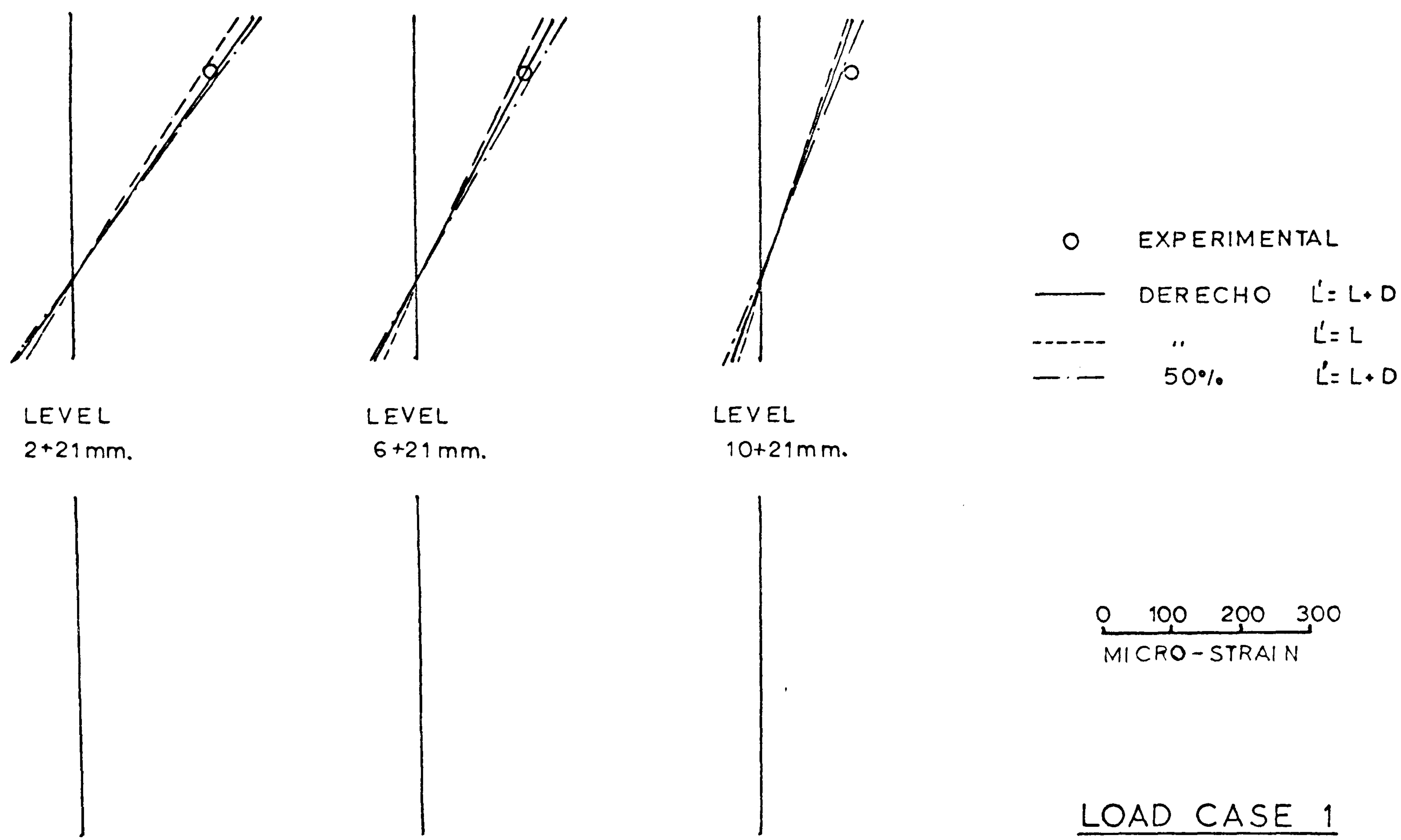
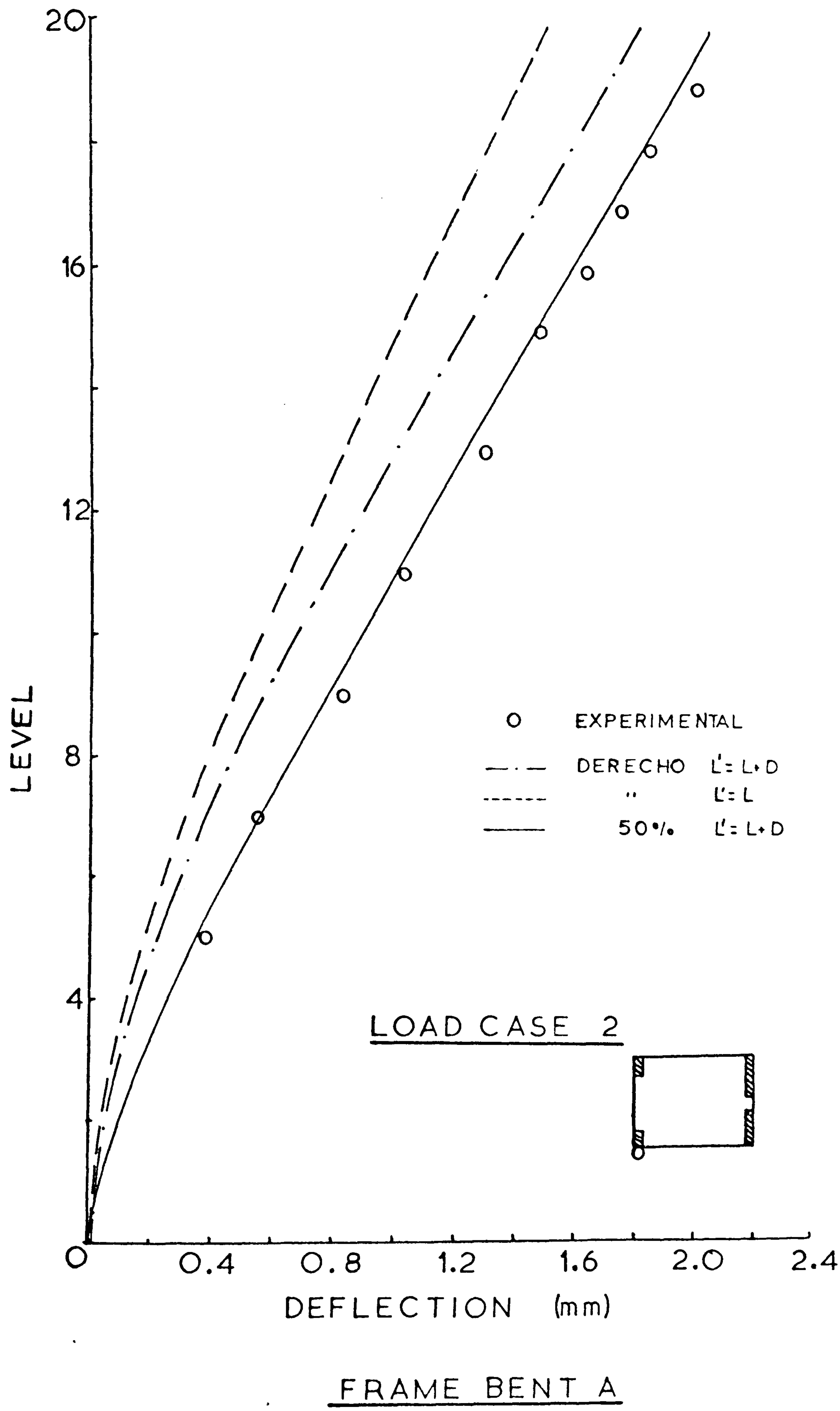

20
18
16
14
12
10
8
6
4
2
LEVEL
O
1.0
2.0
3.0
4.0
5.0
6.0
DEFLECTION (mm.)
EXPERIMENTAL
DERECHO L'= L+D
" L'= L
50% L'= L+D
LOAD CASE 1
FRAME BENT B

Fig. B-2.3



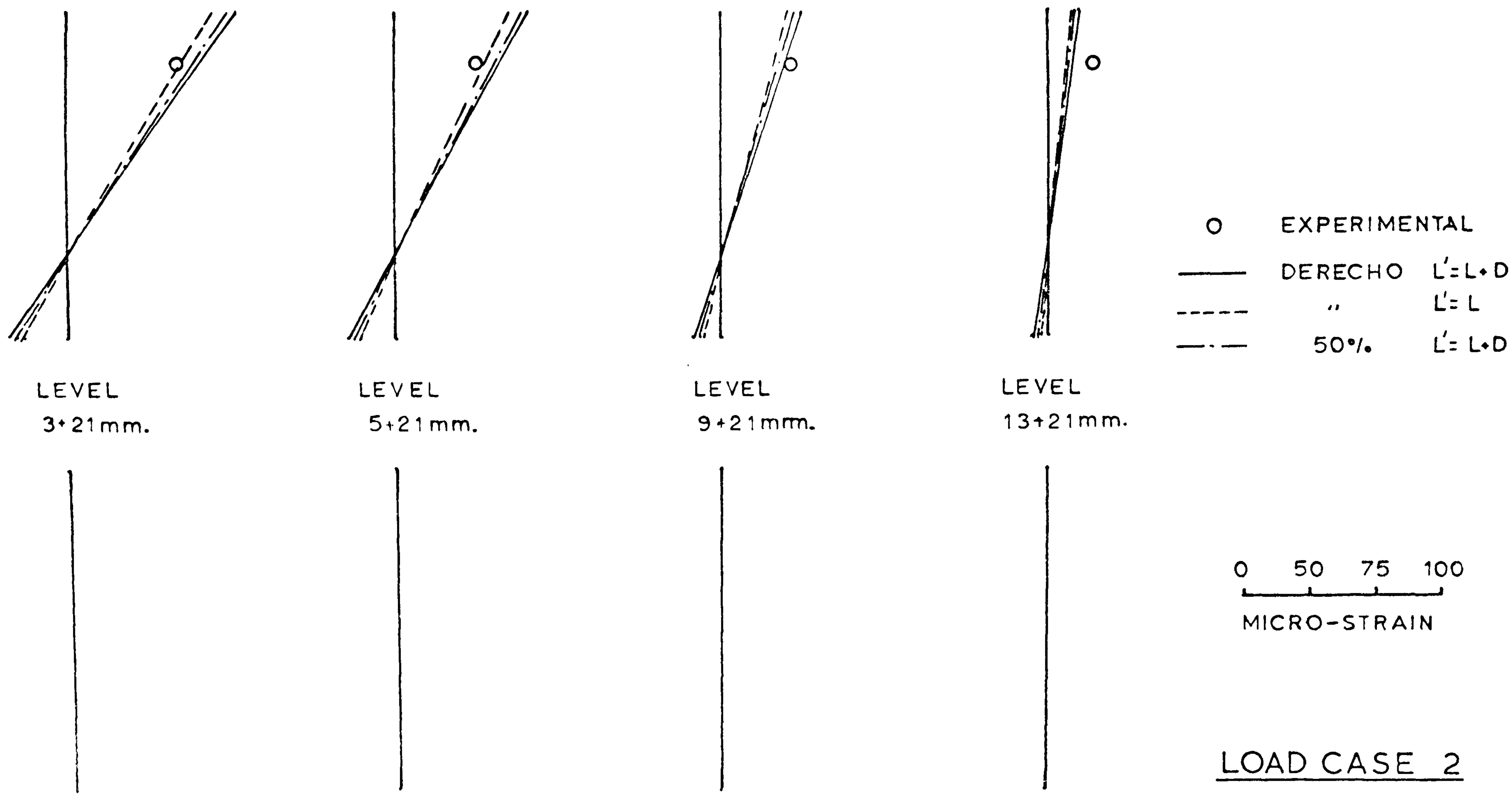
FRAME BENT B

Fig B-2.4



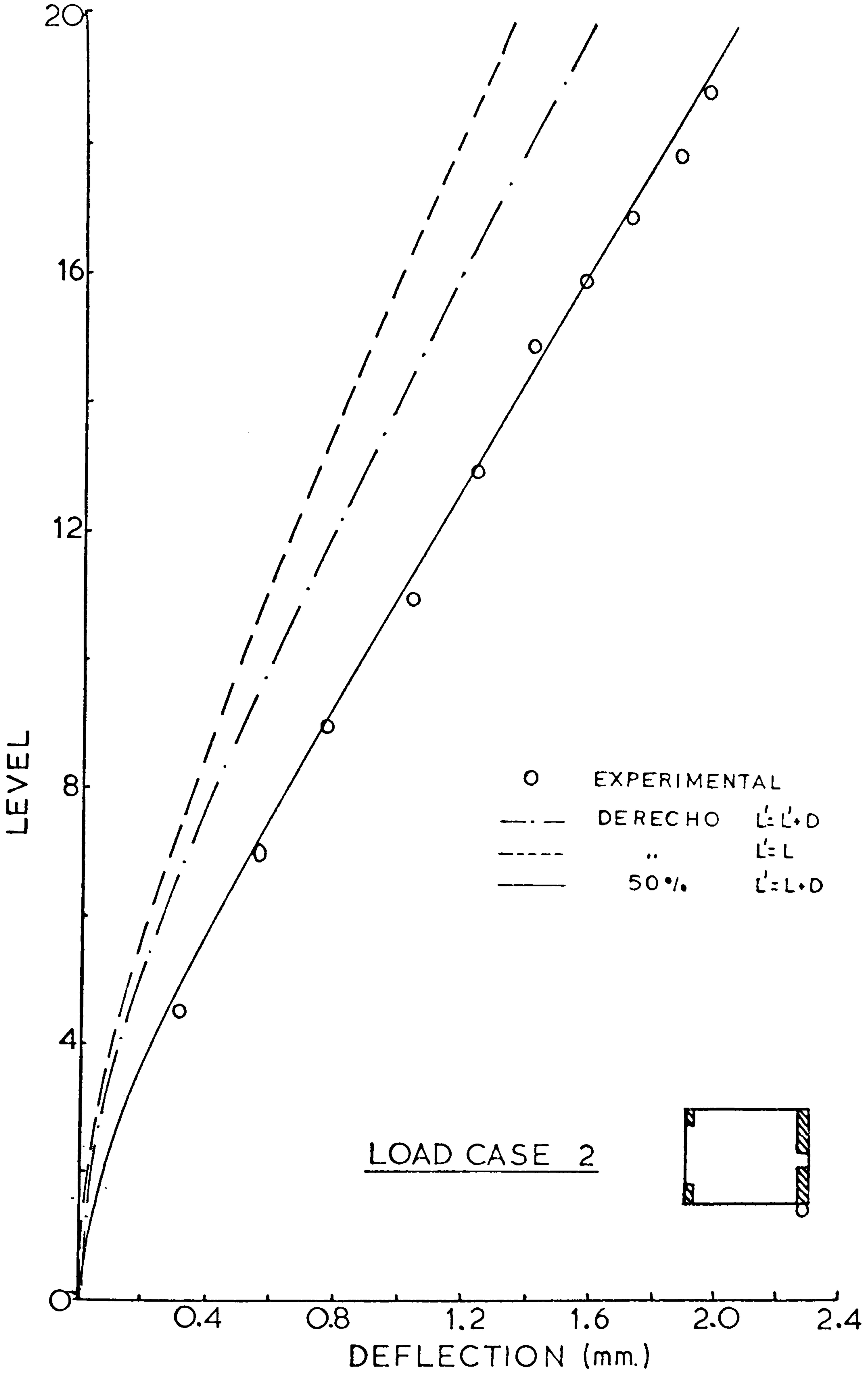
20
16
12
8
4
0
LEVEL
0.4
0.8
1.2
1.6
2.0
2.4
DEFLECTION (mm)
EXPERIMENTAL
DERECHO L'= L+D
" L'= L
50% L'= L+D
LOAD CASE 2
FRAME BENT A

Fig B-2.5



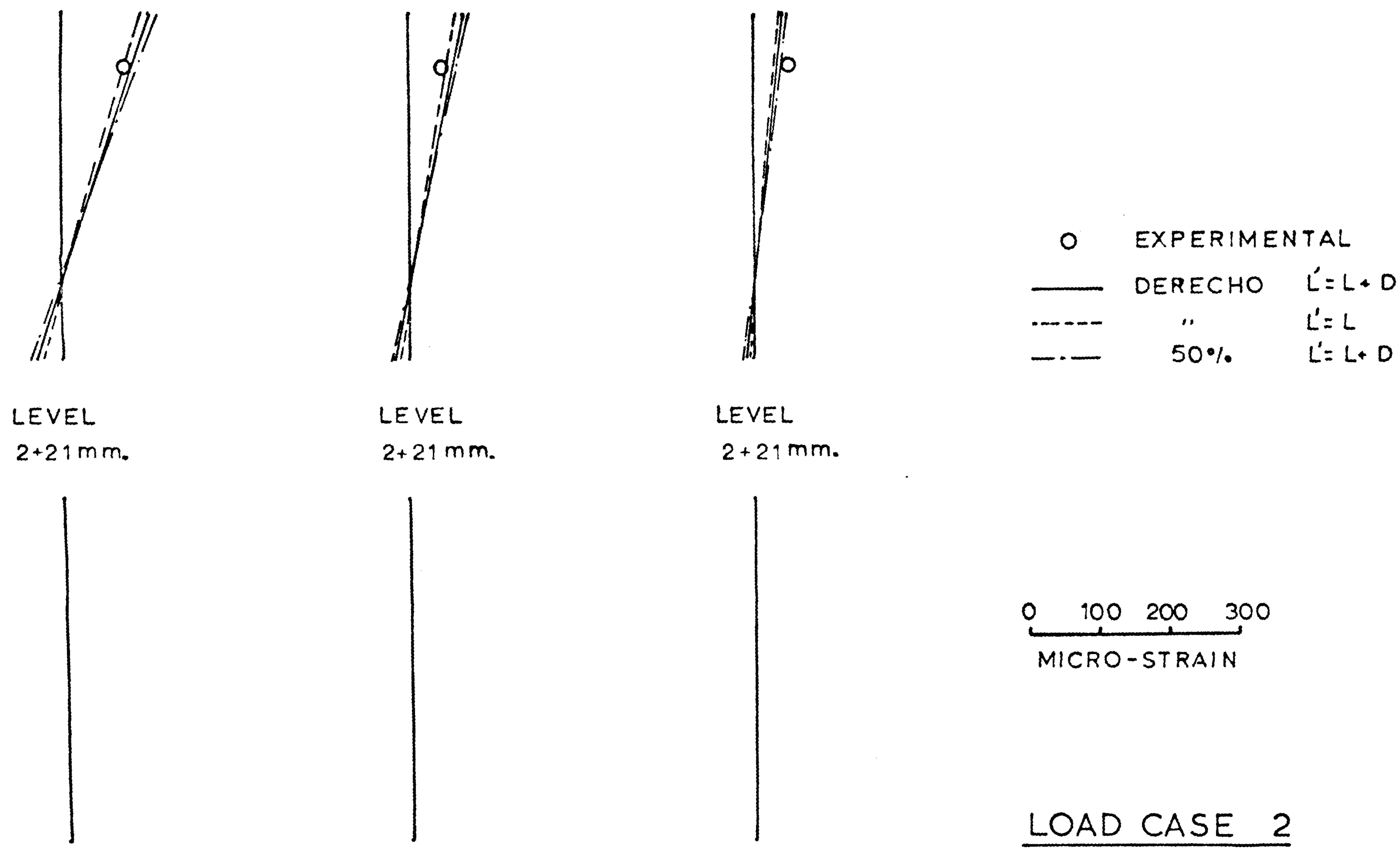
FRAME BENT A

Fig B-2.6



FRAME BENT B

Fig B-2.7



FRAME BENT B

Fig B-2.8

LEVEL OF MEASURED DEFLECTIONS	LEVEL AT WHICH TORQUE APPLIED							
	20		15		10		5	
	FRAME BENT		FRAME BENT		FRAME BENT		FRAME BENT	
	A	B	A	B	A	B	A	B
18	11.97	5.93	6.3	14.44	1.50	6.25	—	—
16	—	—	7.68	8.79	—	—	—	—
15	9.81	3.81	—	—	2.80	8.22	—	—
12	5.69	2.01	8.90	5.88	9.51	8.59	—	—
9	1.40	—	4.28	2.73	7.82	8.49	2.67	2.21
6	—	—	3.09	—	3.50	5.27	6.03	3.69

DEFLECTIONS GIVEN ABOVE,(mm.x10), ARE SCALED LINEARLY TO REPRESENT A TORQUE OF 100 N.mm. AT THE INDICATED LEVELS

Table B-2

MODEL No. 2

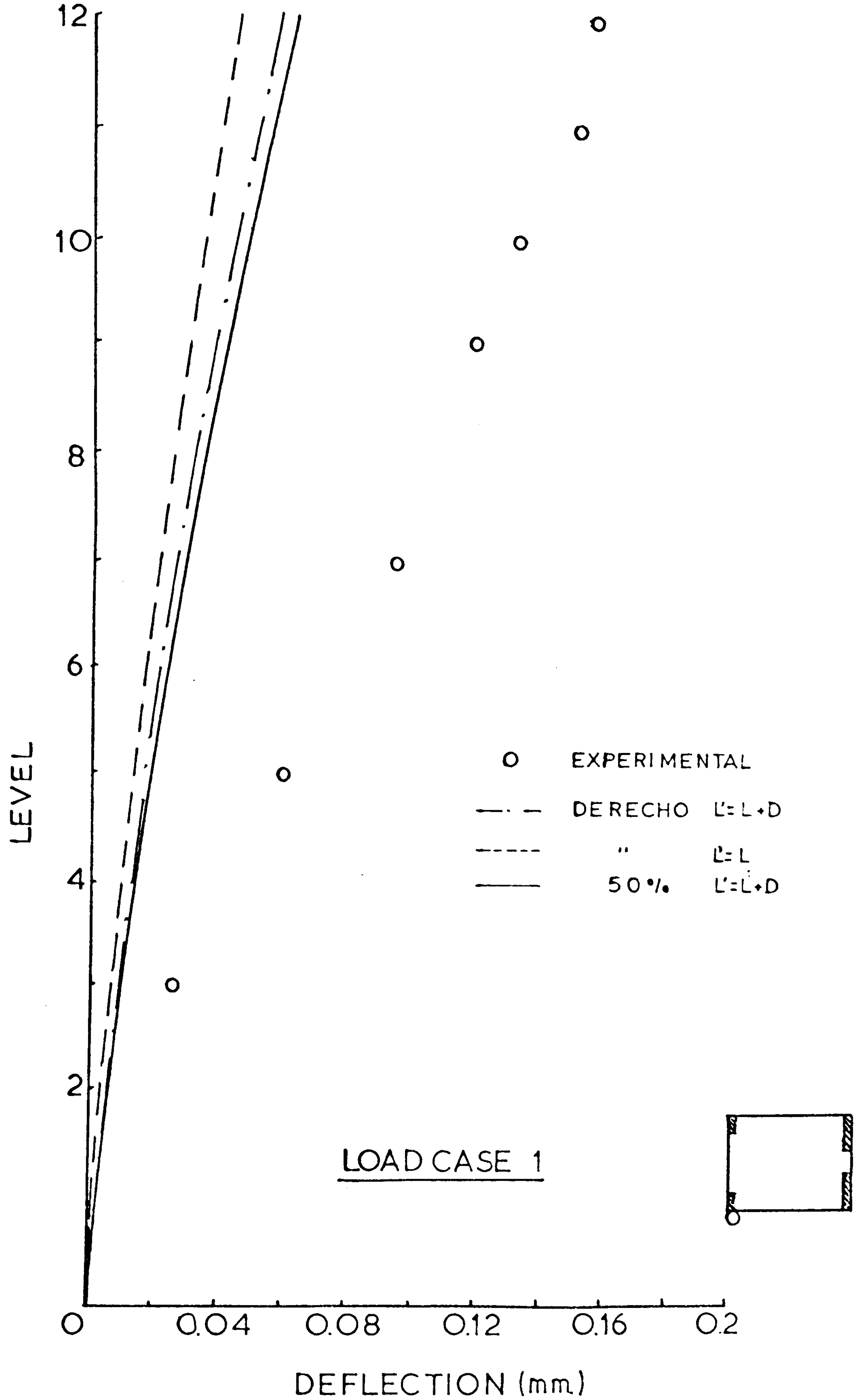
EXPERIMENTAL FUNDAMENTAL FREQUENCY

6.06 Hz

AVERAGE VALUE OF DAMPING

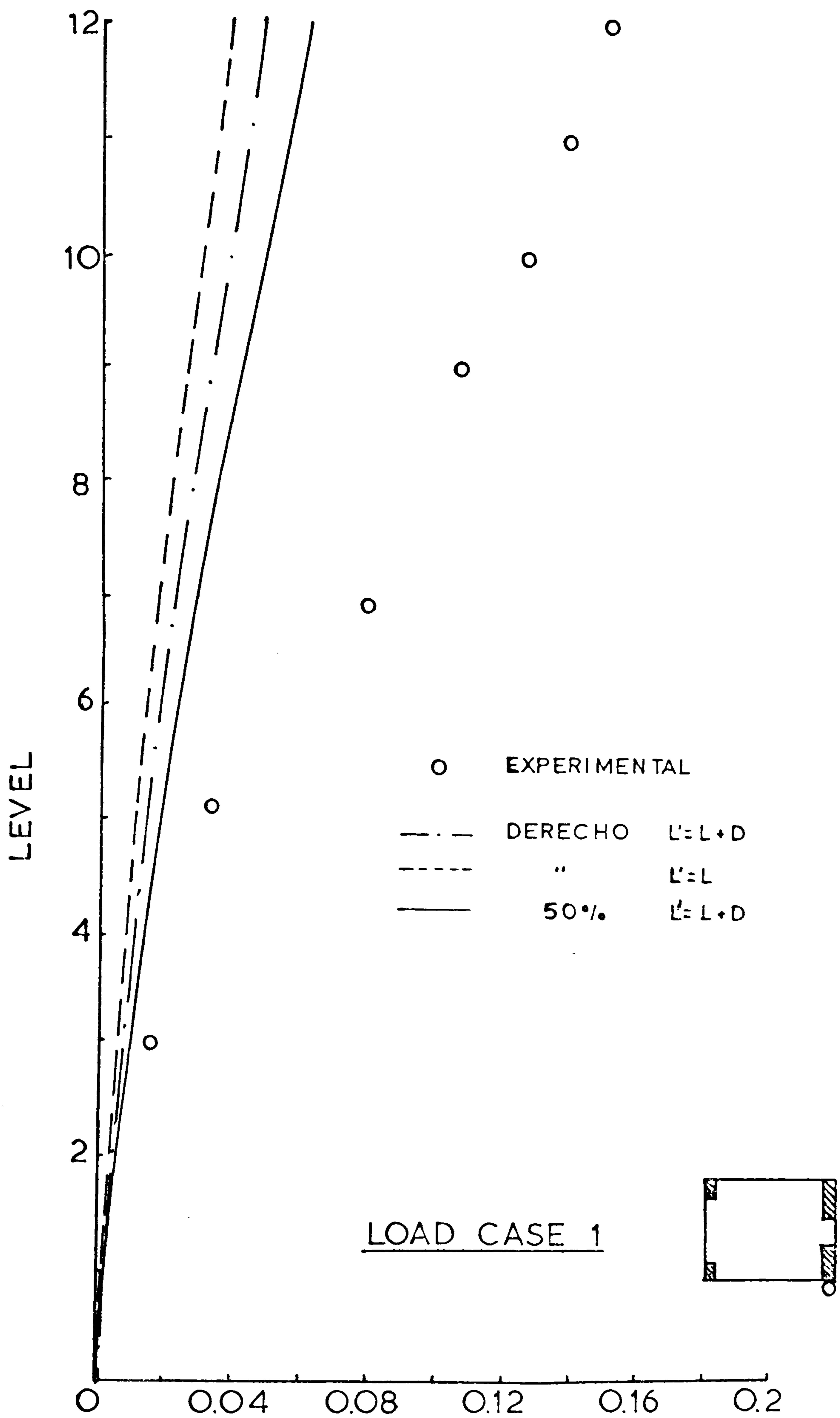
7.3 % Critical

Model No. 3



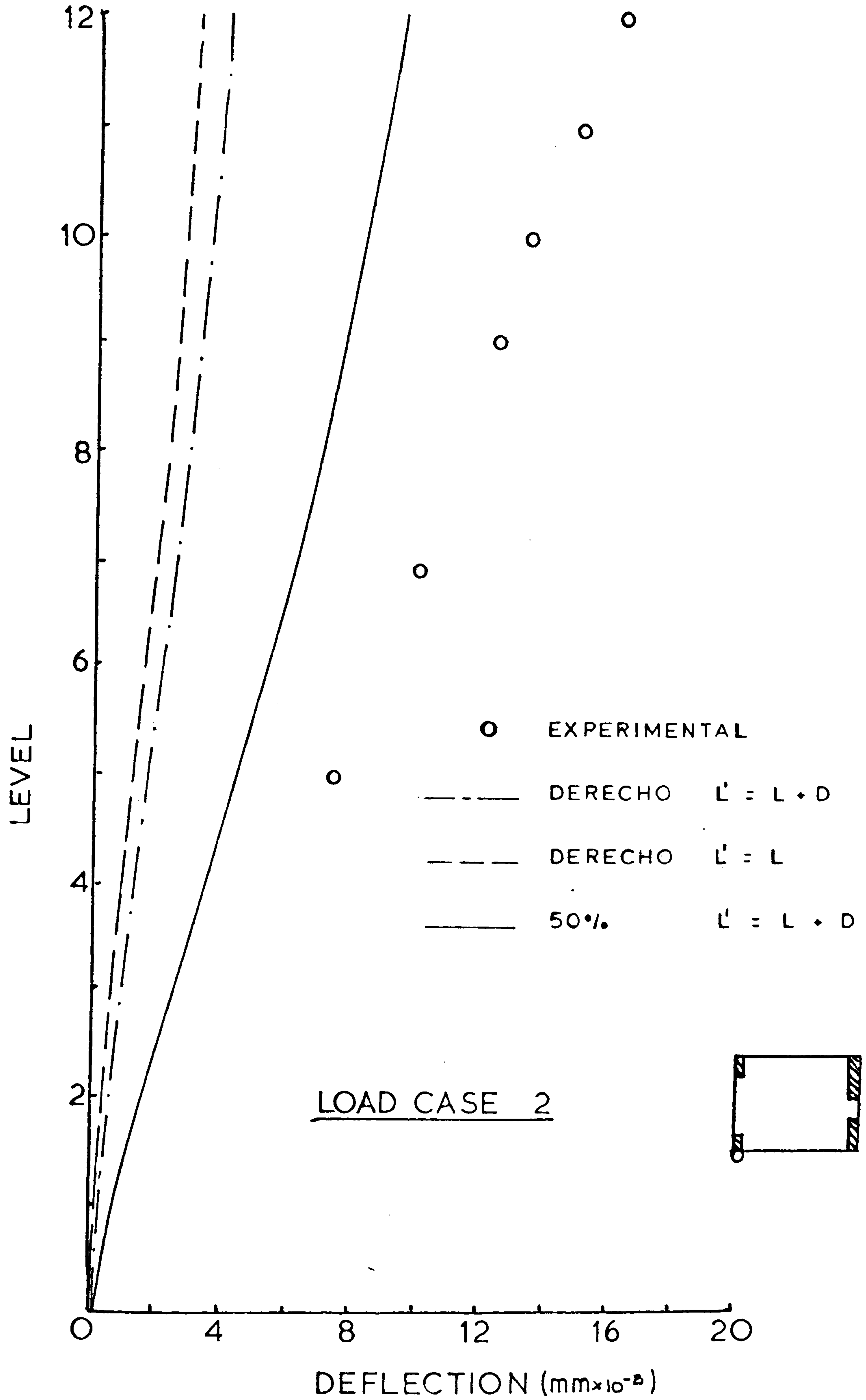
FRAME BENT A

Fig. B-3.1



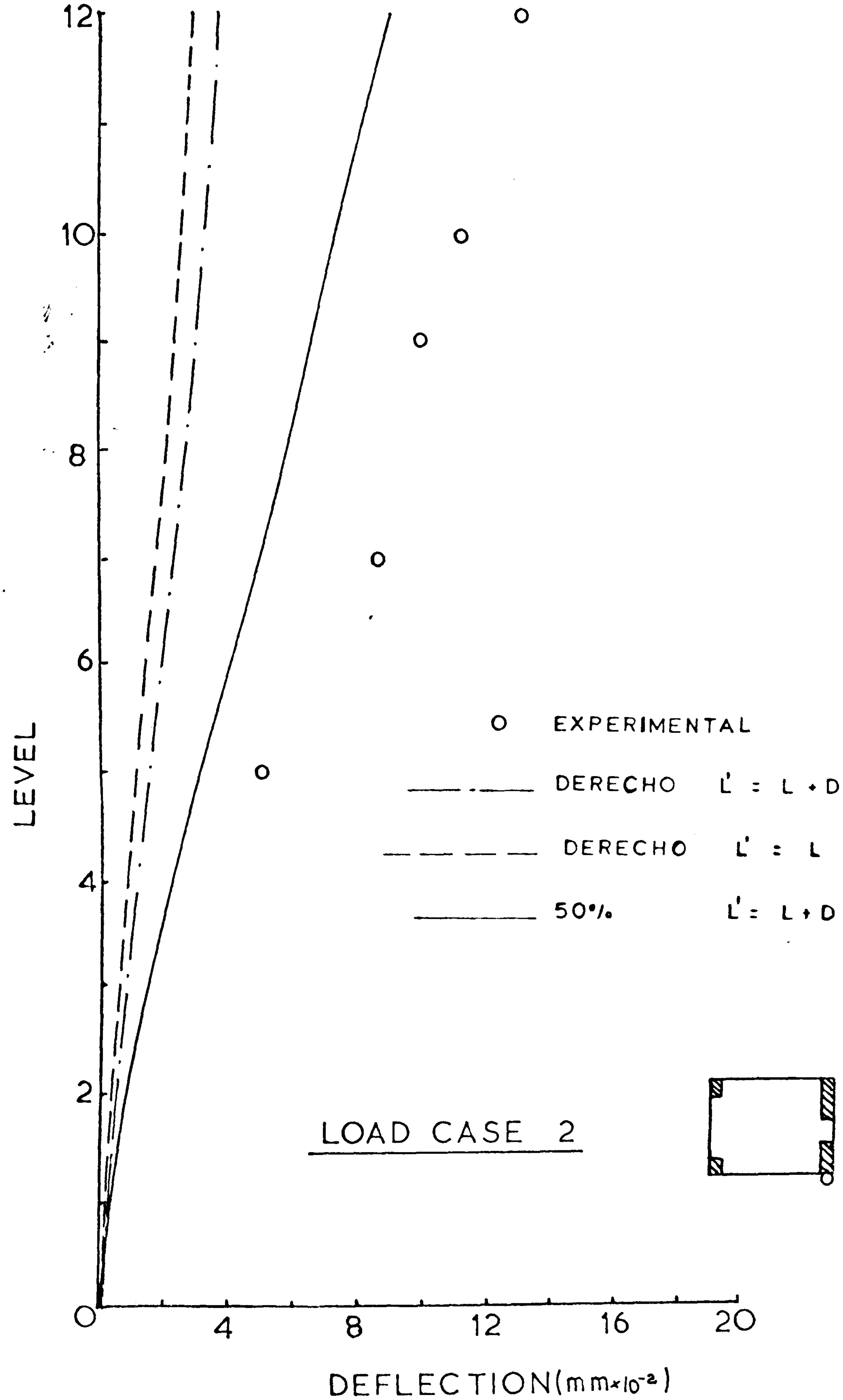
FRAME BENT B

Fig. B-3.2



FRAME BENT A

Fig B-3.3

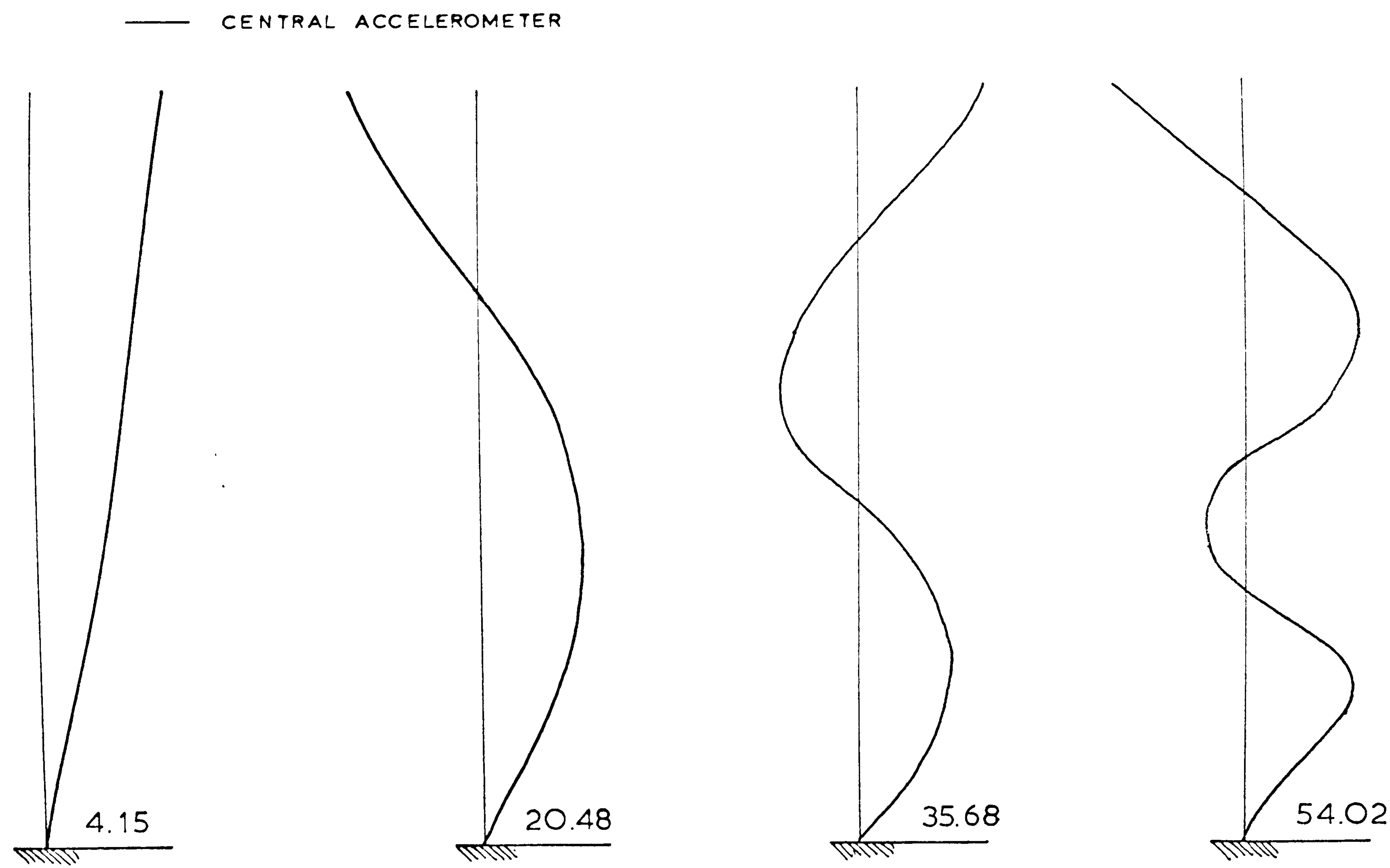


FRAME BENT B

Fig B-3.4

CENTRAL ACCELEROMETER LEVEL 12 PARALLEL TO THE FRAME-BENTS
1ST Bending
2nd Bending
3rd Bending
4th Bending
Test Rig
A
0
10
30
50
70
90
110
130
FREQUENCY (Hz)
A = AMPLITUDE
MODEL No.3 DYNAMIC RESPONSE CURVE

Fig B-3.5



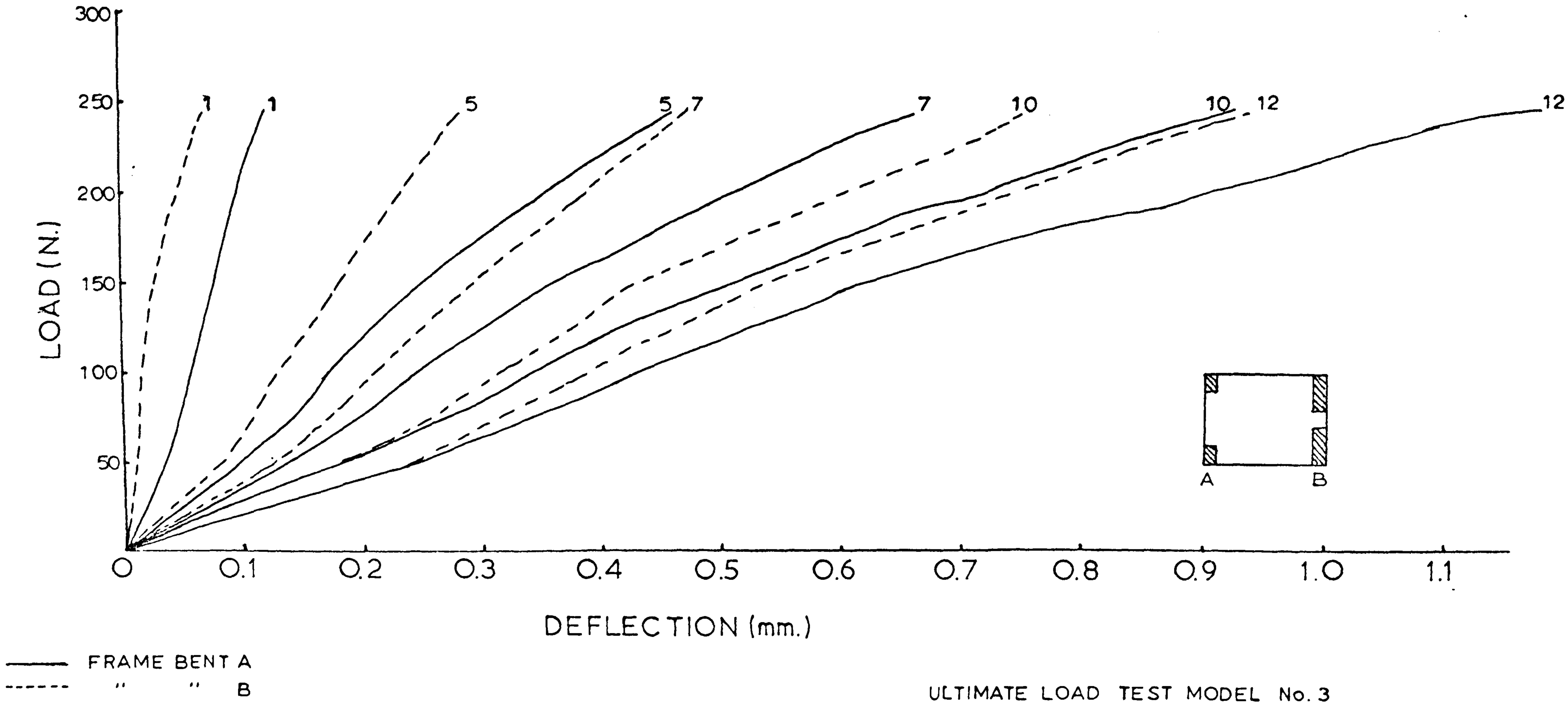
TRANSLATIONAL — MODE SHAPES AND NATURAL FREQ (Hz.)

Fig B-3.6

NATURAL FREQ.(Hz)	MODE TYPE	% CRITICAL DAMPING	
EXPERIMENTAL		FORCED VIBRATION	FREE VIBRATION
4.15	Translation	7.6	6.13
20.48	Translation	5.2	
35.68	Translation	3.2	
54.02	Translation	1.9	

NATURAL FREQUENCIES, MODE TYPES AND DAMPING VALUES FOR MODEL 3

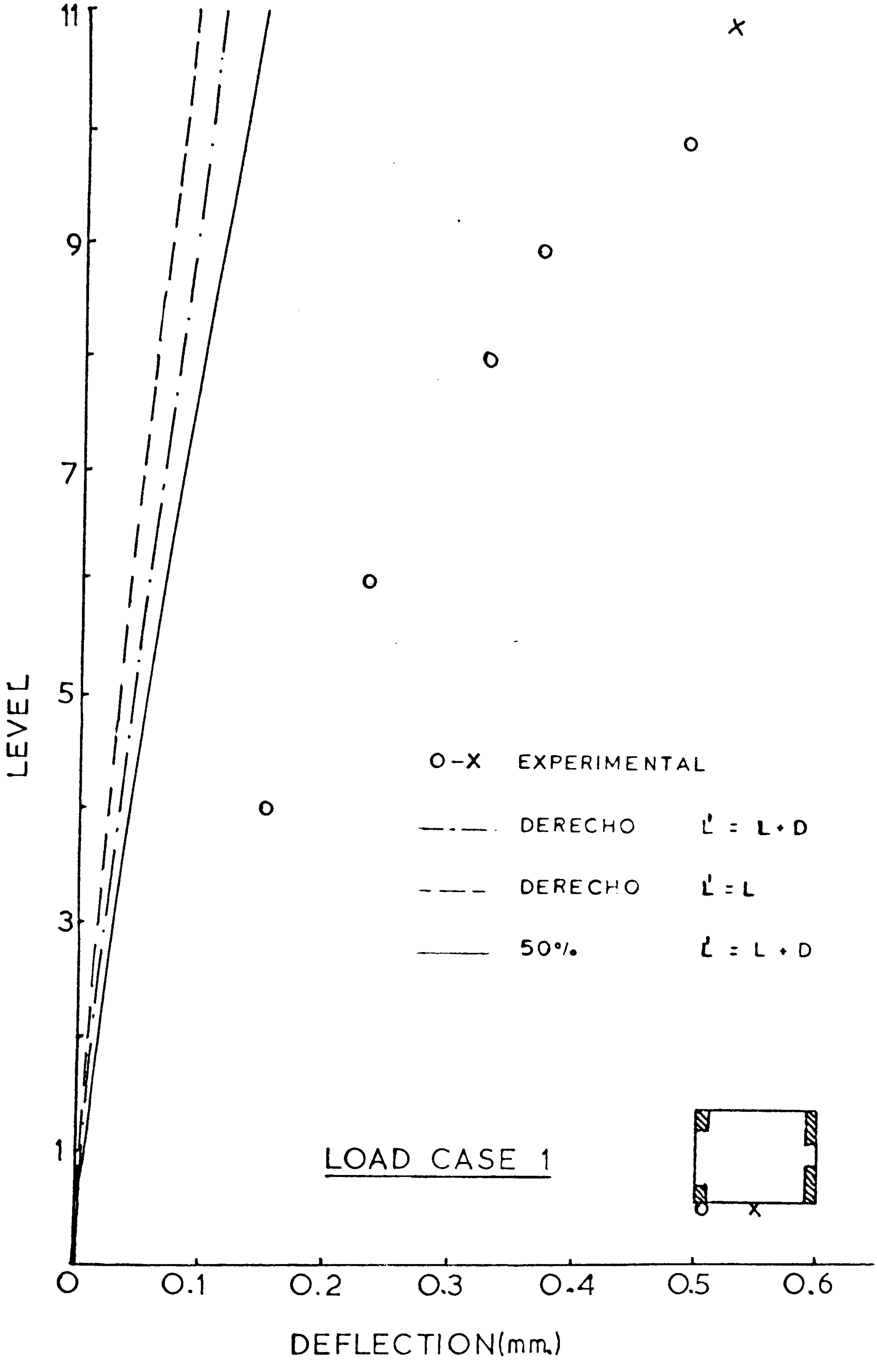
Table B-3



ULTIMATE LOAD TEST MODEL No. 3

Fig. B-3.7

Model No. 4



FRAME BENT A

Fig B-4.1

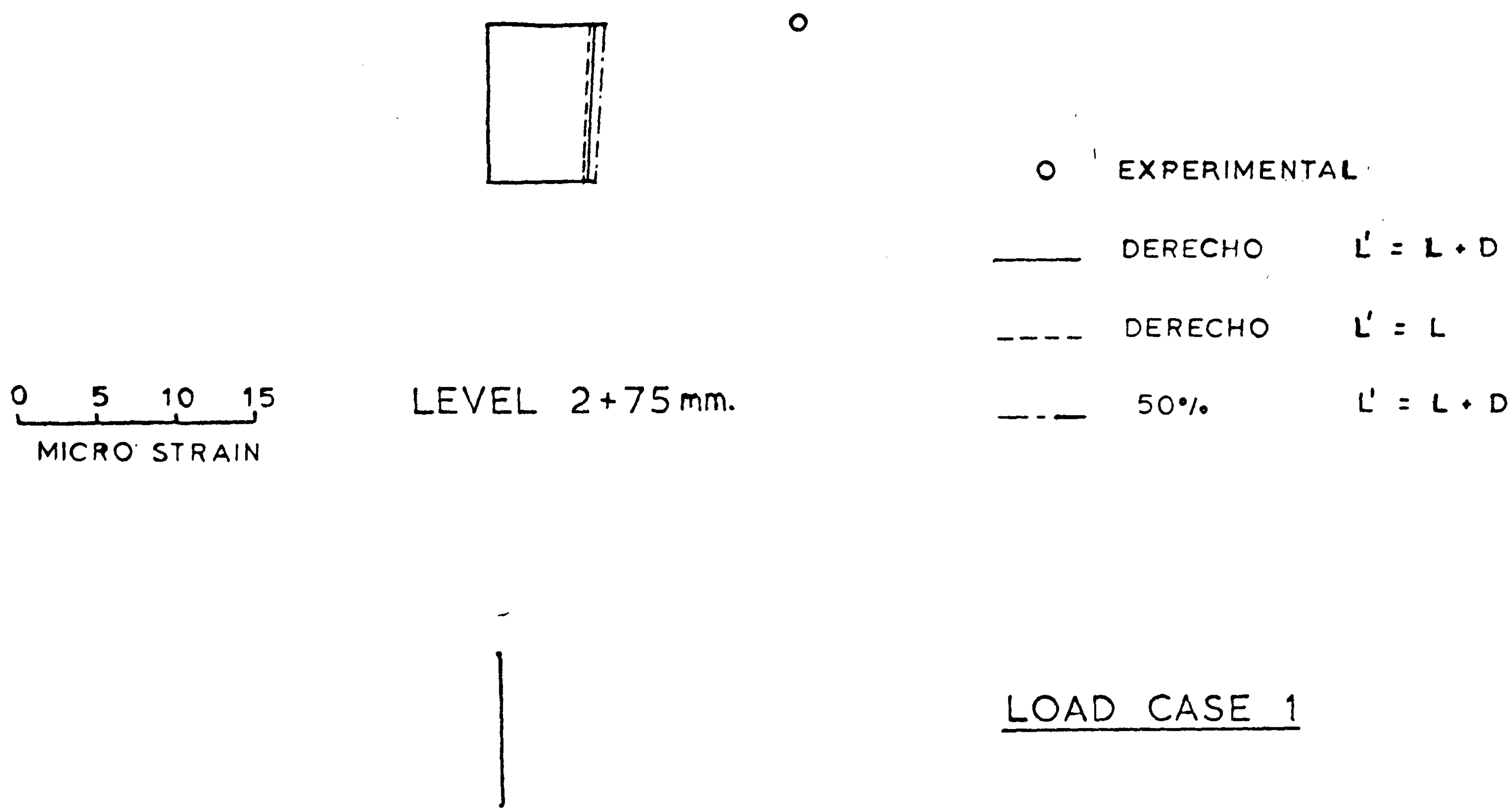
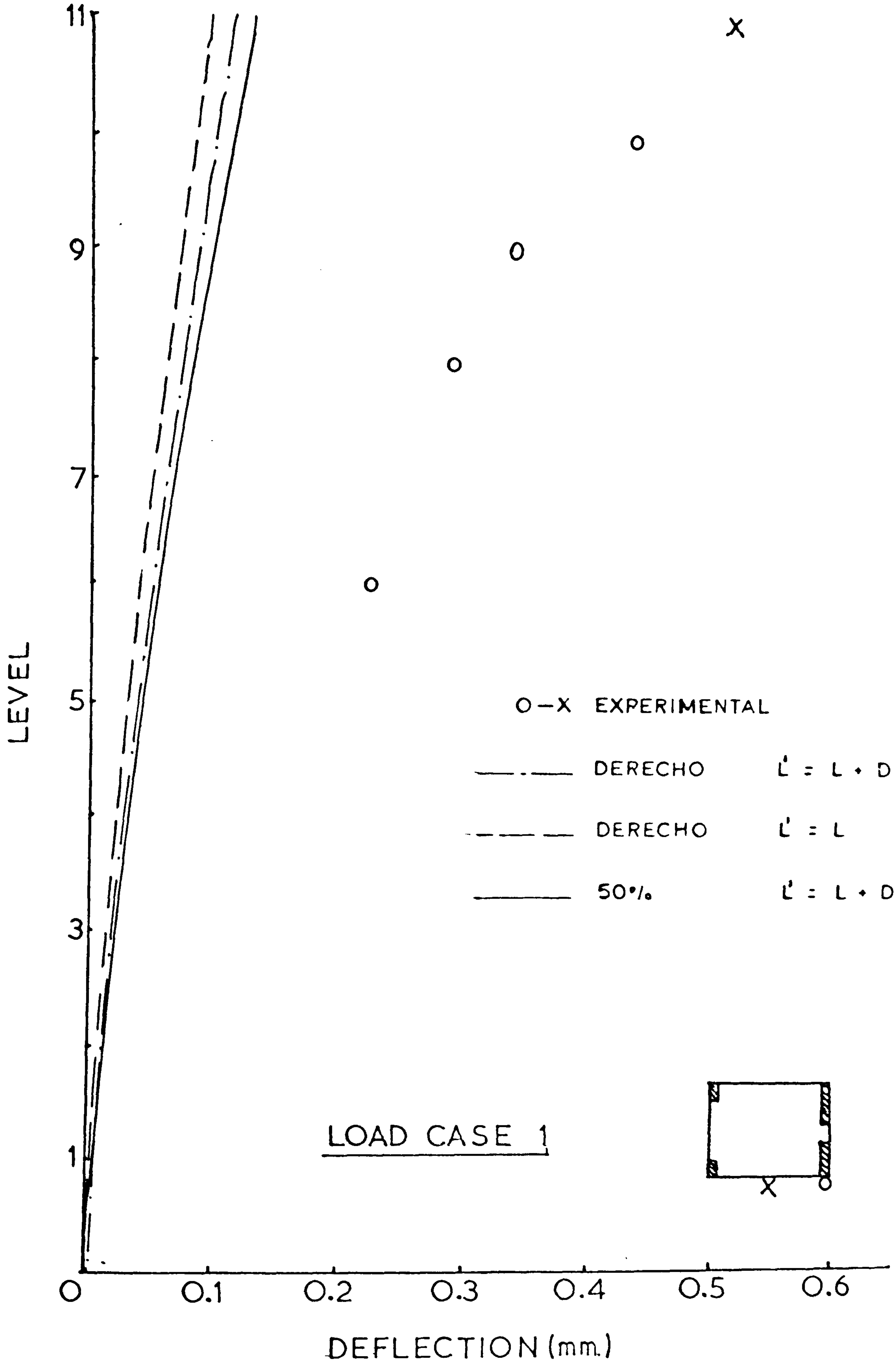
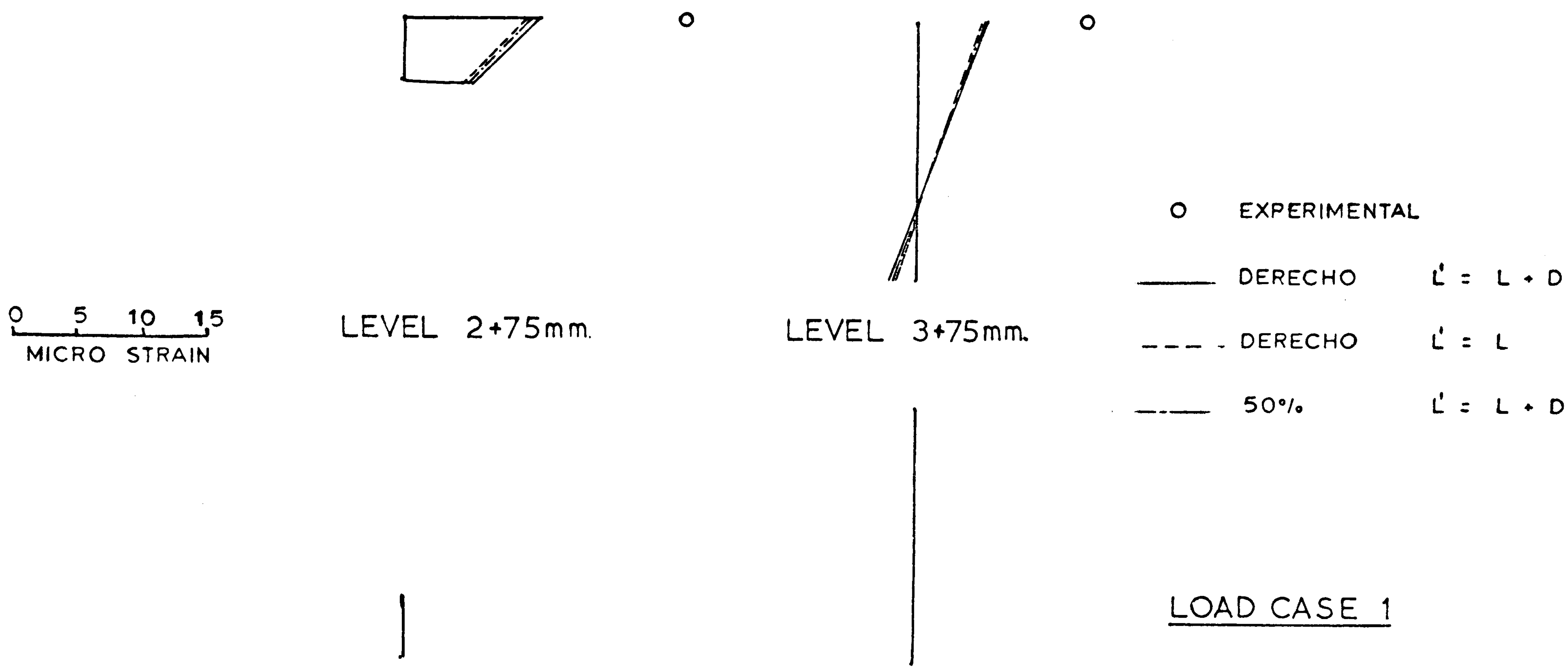

0 5 10 15
MICRO STRAIN
LEVEL 2+75 mm.
o EXPERIMENTAL
DERECHO L' = L + D
DERECHO L' = L
50% L' = L + D
LOAD CASE 1

Fig B-4.2



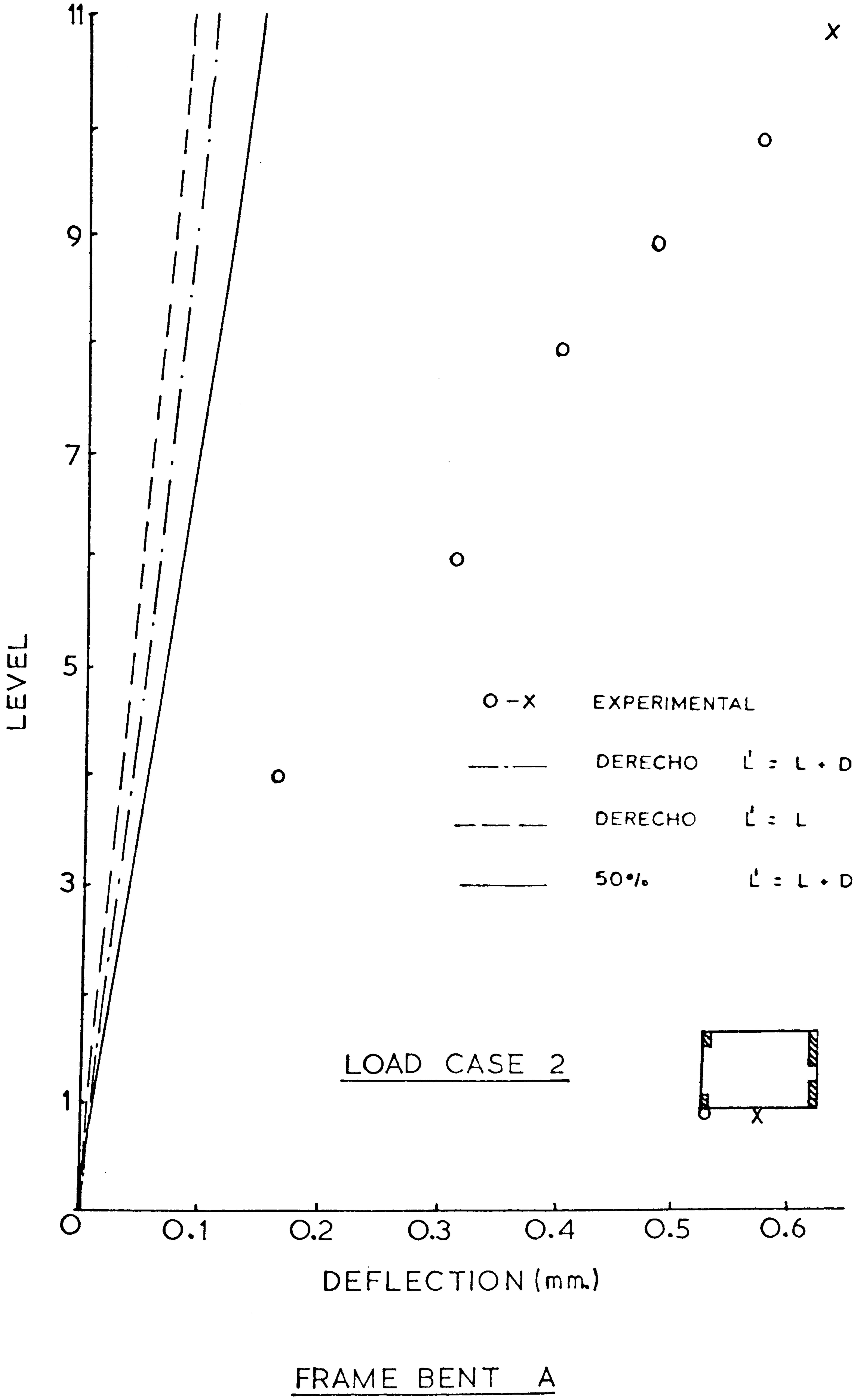
FRAME BENT B

Fig B-4.3



0 5 10 15
MICRO STRAIN
LEVEL 2+75mm.
LEVEL 3+75mm.
EXPERIMENTAL
DERECHO L' = L + D
DERECHO L' = L
50% L' = L + D
LOAD CASE 1

Fig B-4.4



LEVEL
DEFLECTION (mm.)
O – X EXPERIMENTAL
DERECHO L' = L + D
DERECHO L' = L
50% L' = L + D
LOAD CASE 2
FRAME BENT A

Fig B-4.5

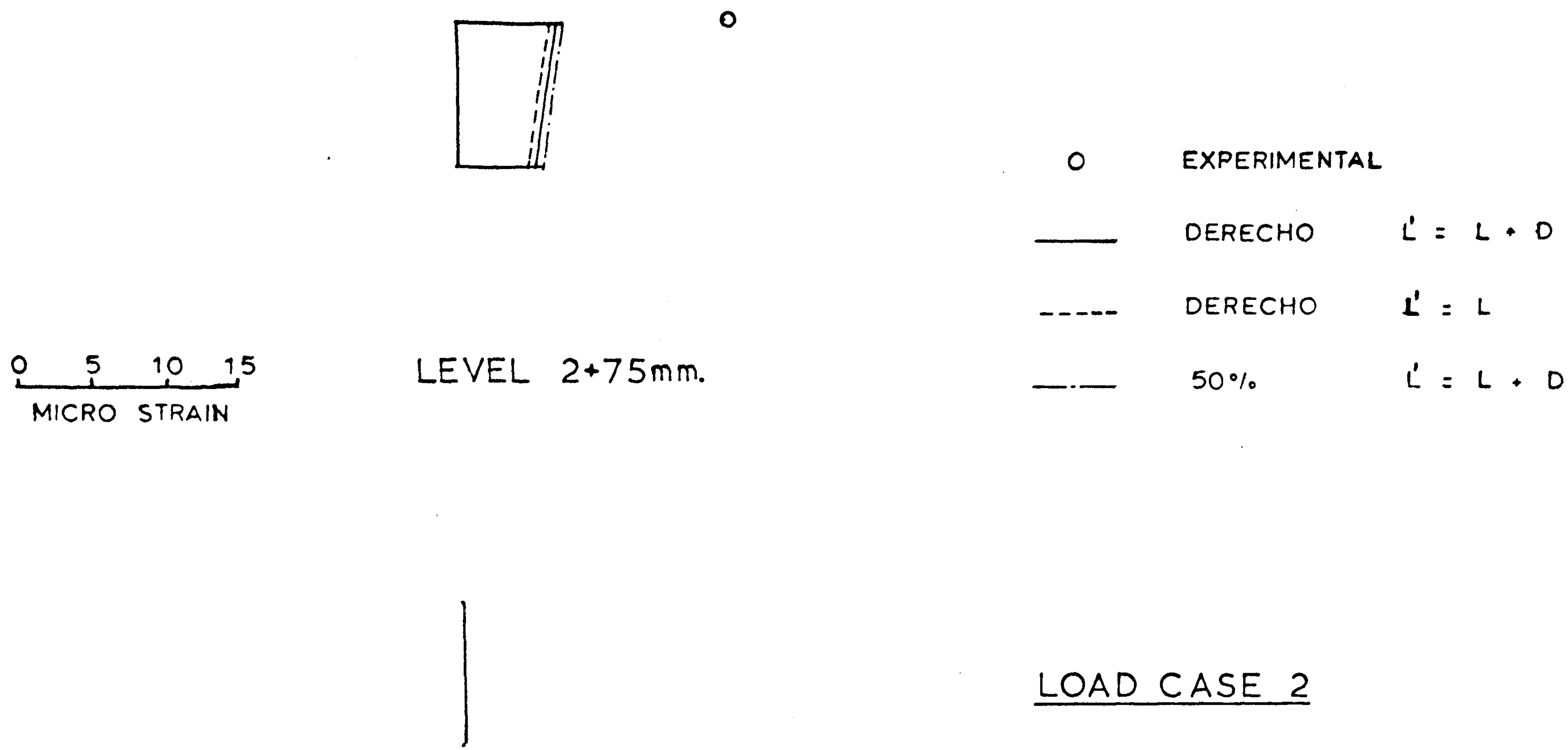
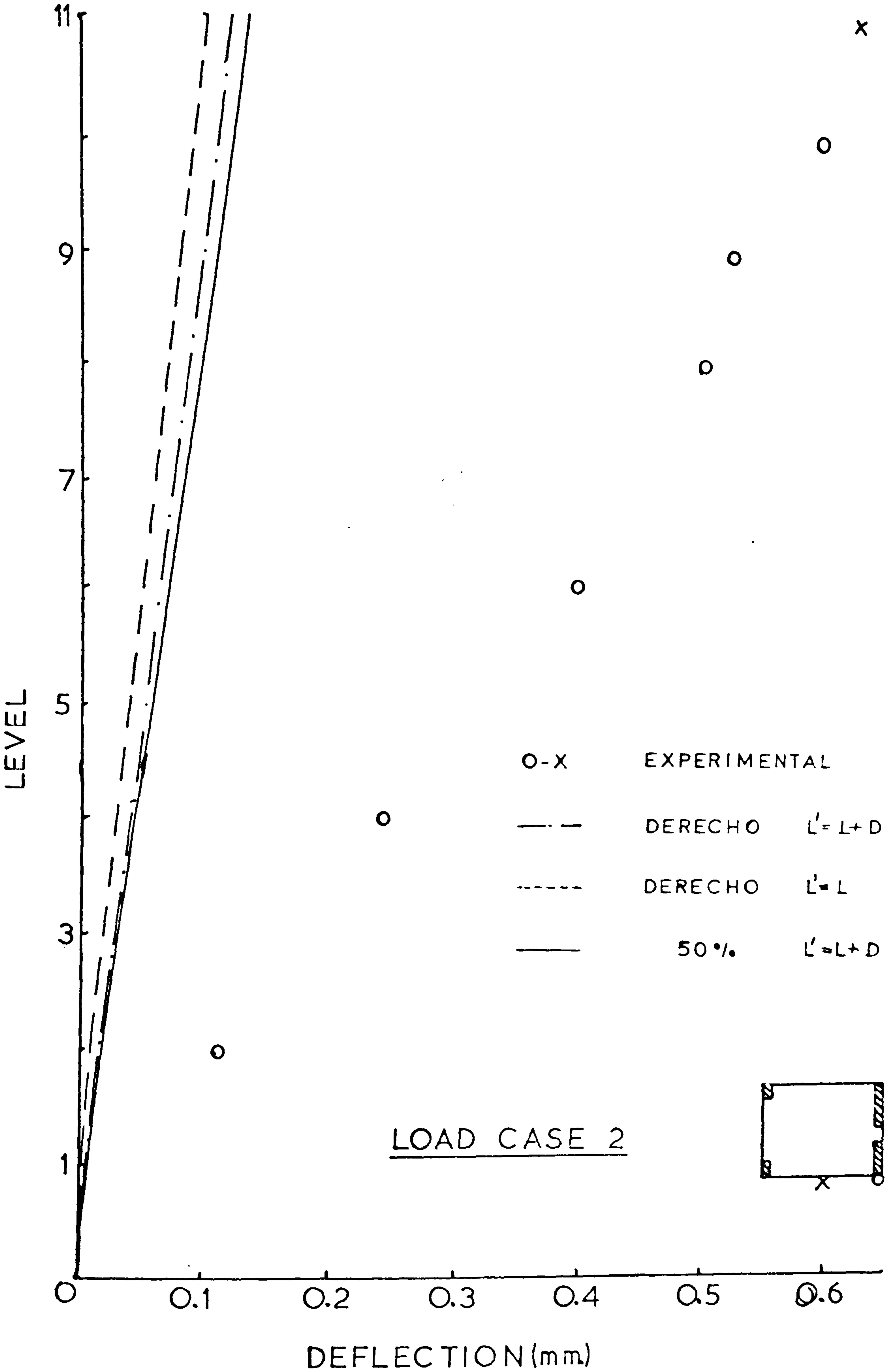

0 5 10 15
MICRO STRAIN
LEVEL 2+75mm.
O EXPERIMENTAL
DERECHO L' = L + D
DERECHO L' = L
50% L' = L + D
LOAD CASE 2

Fig B-4.6



FRAME BENT B

Fig B-4.7

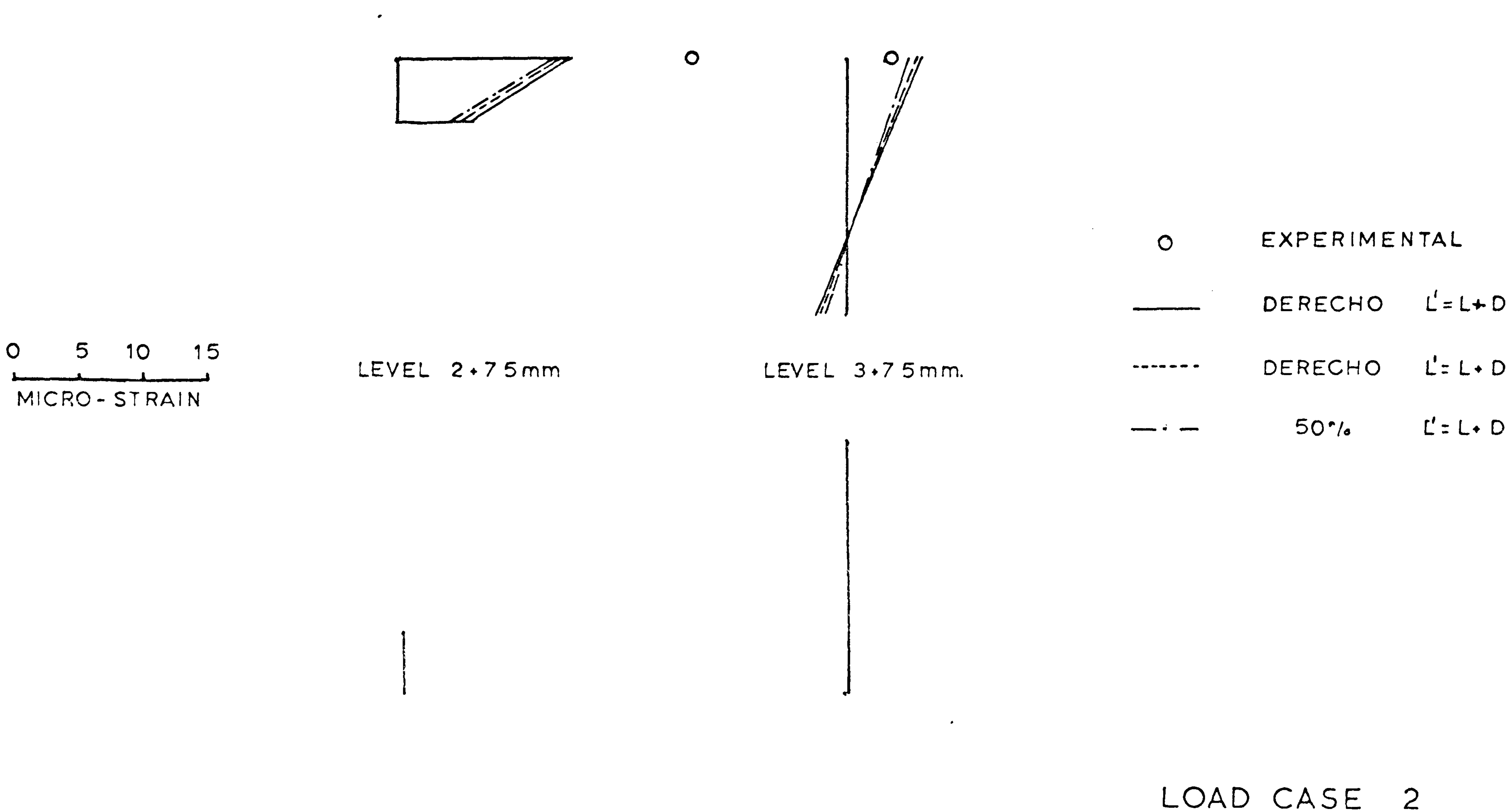
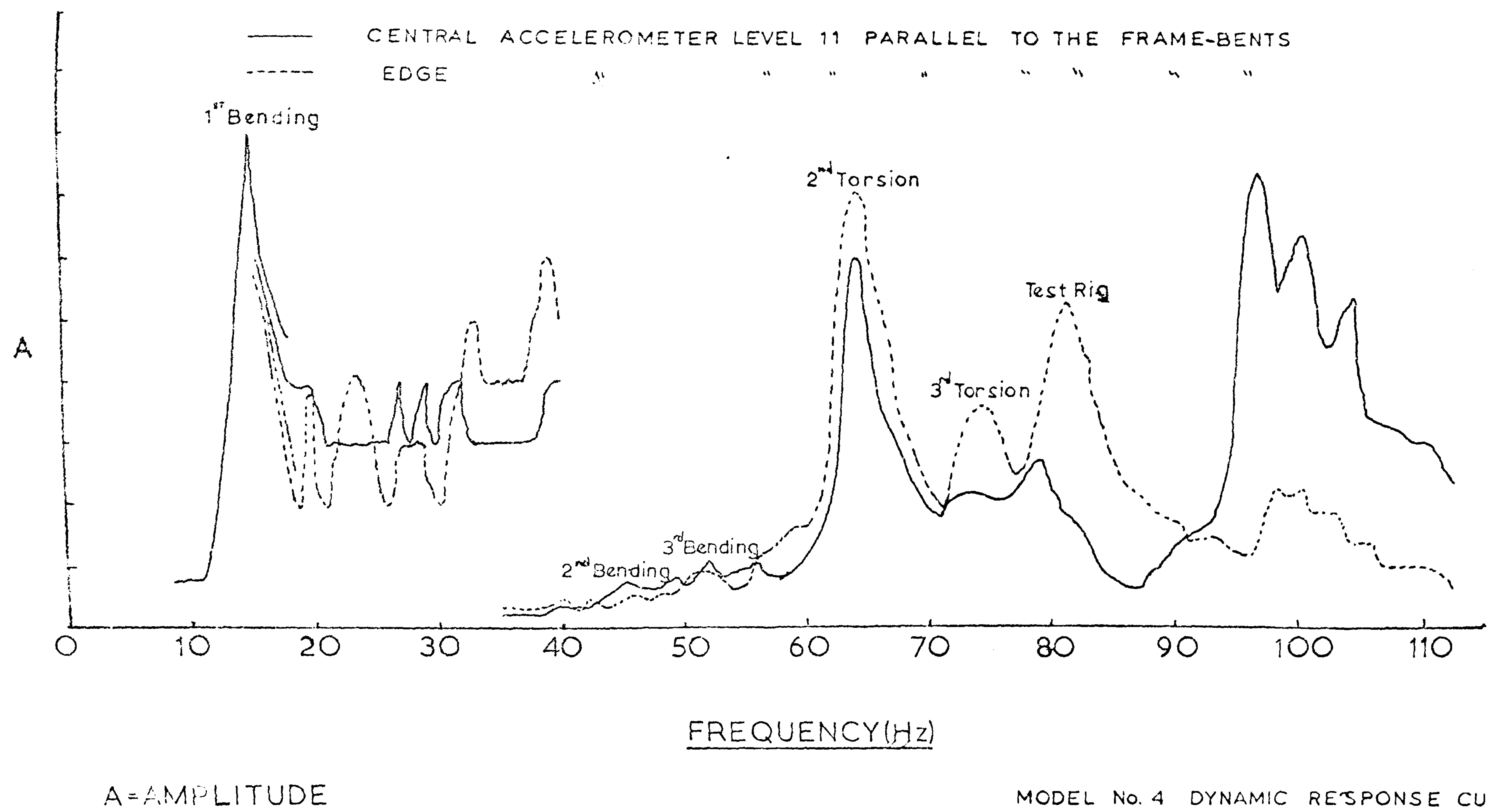
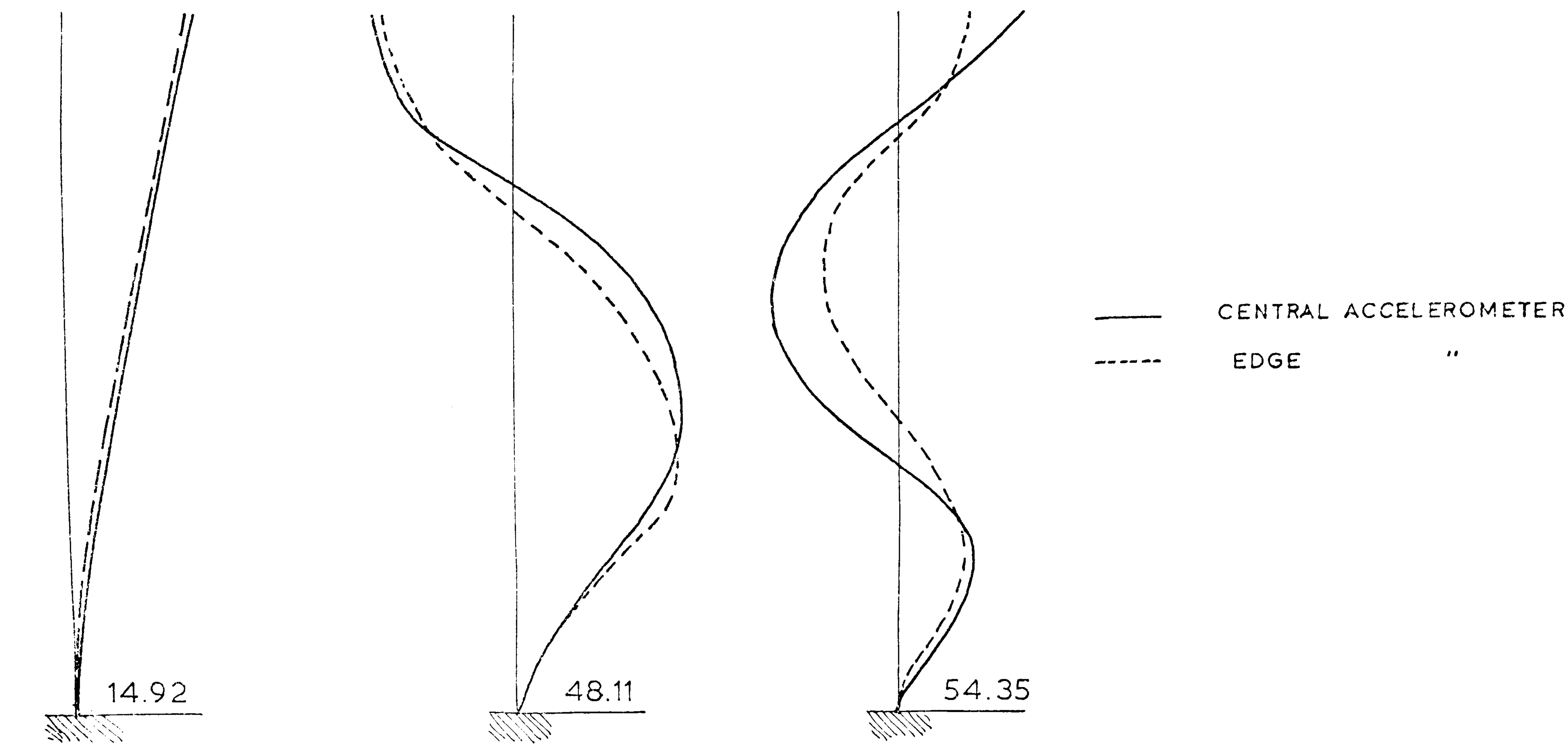

0 5 10 15
MICRO - STRAIN
LEVEL 2·75mm
LEVEL 3·75mm.
EXPERIMENTAL
DERECHO L' = L + D
DERECHO L' = L + D
50% L' = L + D
LOAD CASE 2

Fig B-4.8



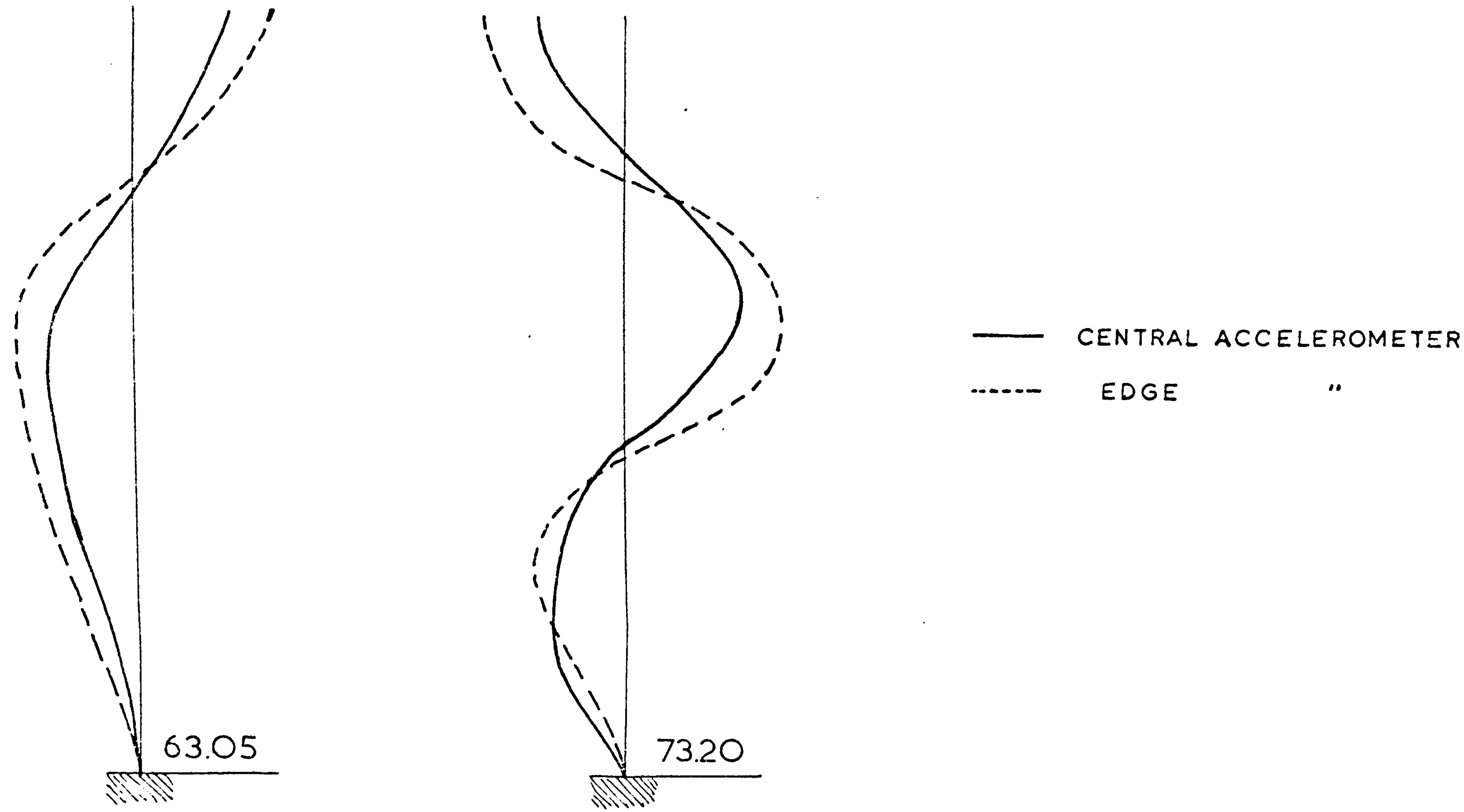
MODEL No. 4 DYNAMIC RESPONSE CURVE

Fig B-4.9



TRANSLATIONAL — MODE SHAPES AND NATURAL FREQ. (Hz)

Fig B-4.10



ROTATIONAL-MODE SHAPES AND NATURAL FREQ.(Hz)

Fig B-4.11

NATURAL FREQ.(Hz.)	MODE TYPE	% CRITICAL DAMPING
EXPERIMENTAL		FORCED VIBRATION
14.93	Translation	2.26
48.11	Translation	0.28
54.35	Translation	1.48
63.05	Rotation	0.77
73.20	Rotation	0.94

NATURAL FREQUENCIES, MODE TYPES AND DAMPING VALUES FOR MODEL 4

Table B-4

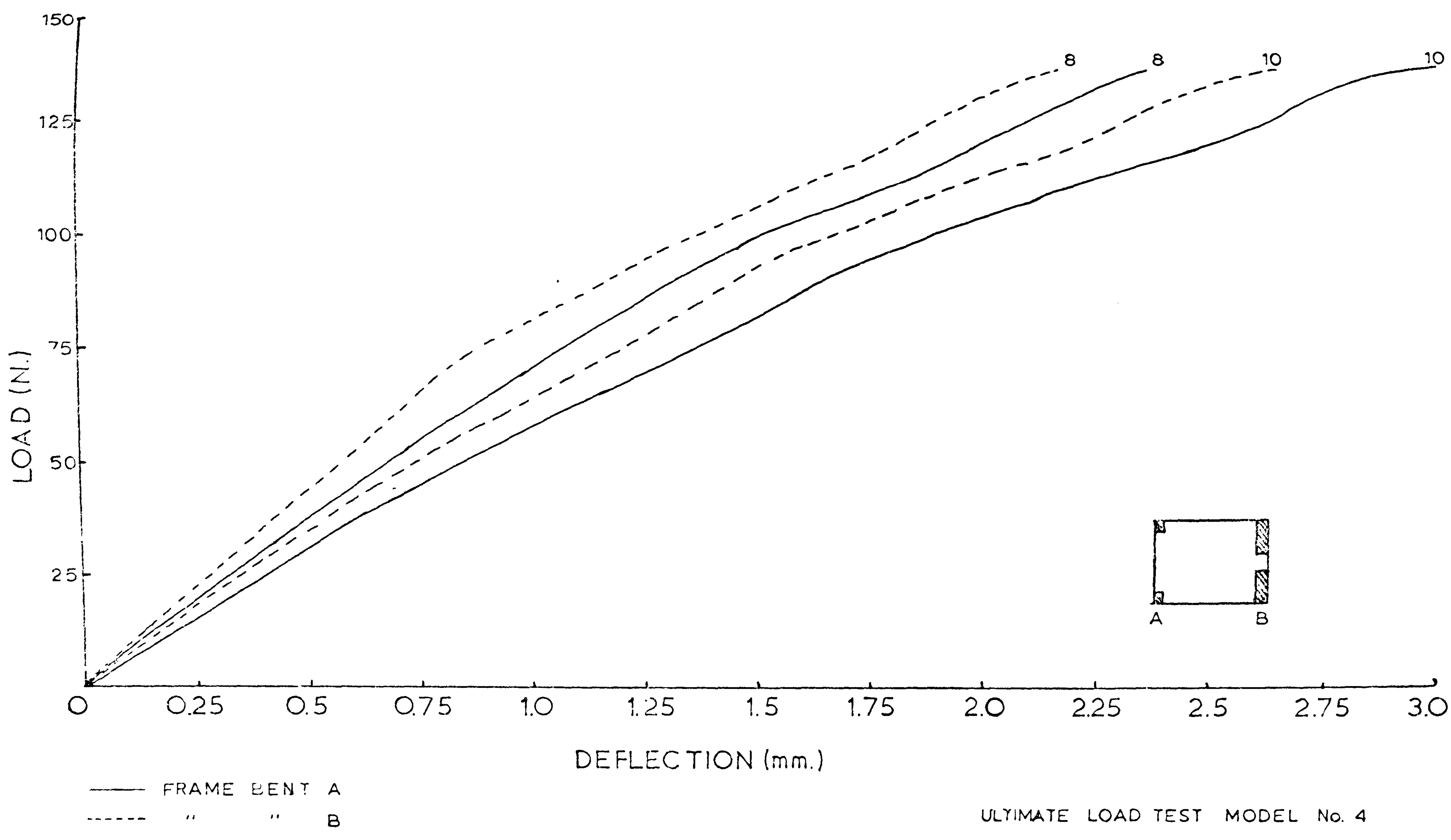

150
125
100
75
50
25
LOAD (N.)
8
8
10
10
A
B
0
0.25
0.5
0.75
1.0
1.25
1.5
1.75
2.0
2.25
2.5
2.75
3.0
DEFLECTION (mm.)
FRAME BENT A
" " B
ULTIMATE LOAD TEST MODEL No. 4

Fig. B-4.12

Model No. 5

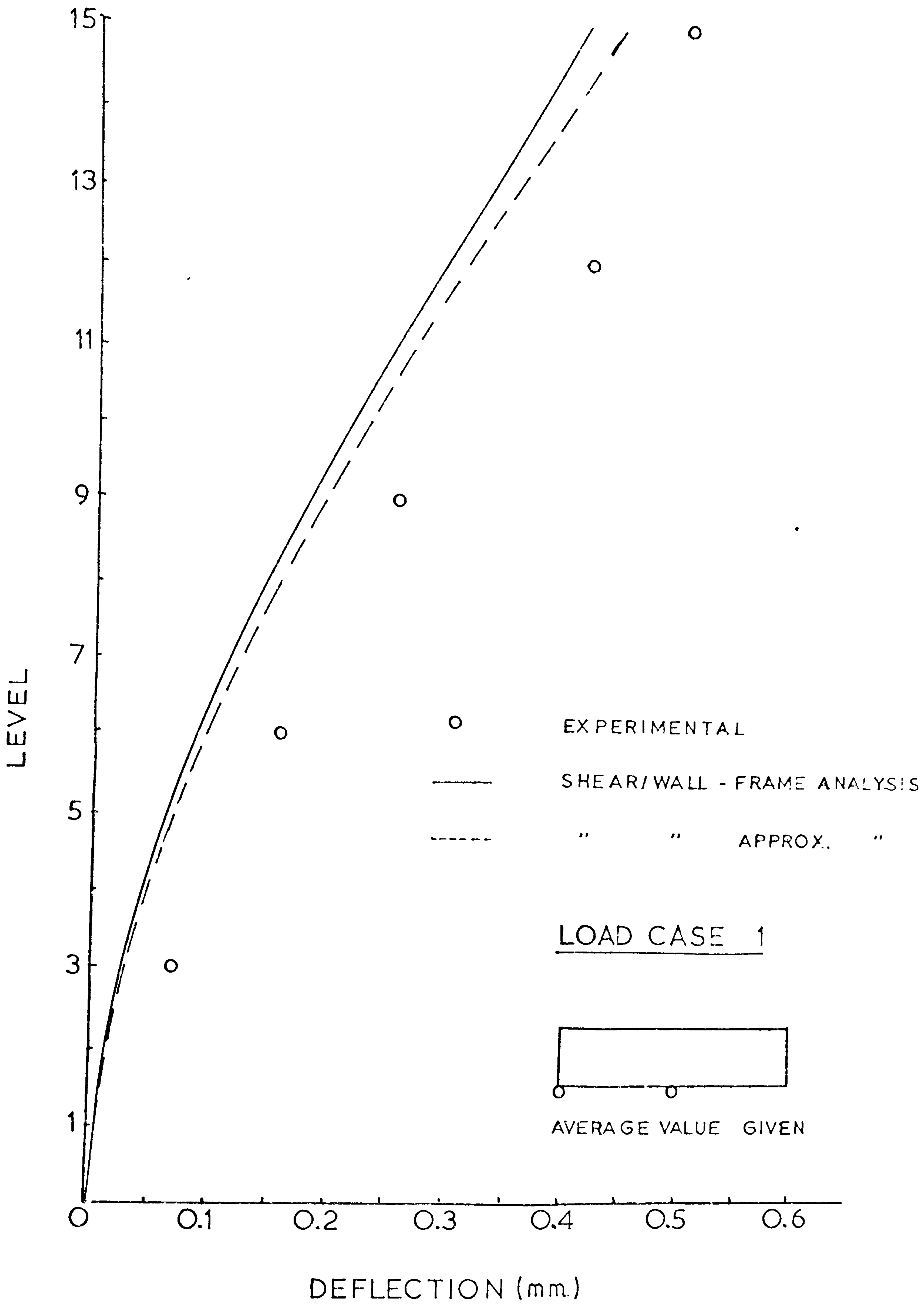

15
13
11
9
7
5
3
1
LEVEL
O
0.1
0.2
0.3
0.4
0.5
0.6
DEFLECTION (mm.)
EXPERIMENTAL
SHEAR/WALL - FRAME ANALYSIS
" " APPROX. "
LOAD CASE 1
AVERAGE VALUE GIVEN

Fig.B-5.1


× EXPERIMENTAL LEVEL 2+10 mm.
○ " " 1+10 mm.
THEORETICAL " 2+10 mm
" " 1+10 mm
0 40 80 120
MICRO STRAIN
LOAD CASE 1

Fig. B-5.2

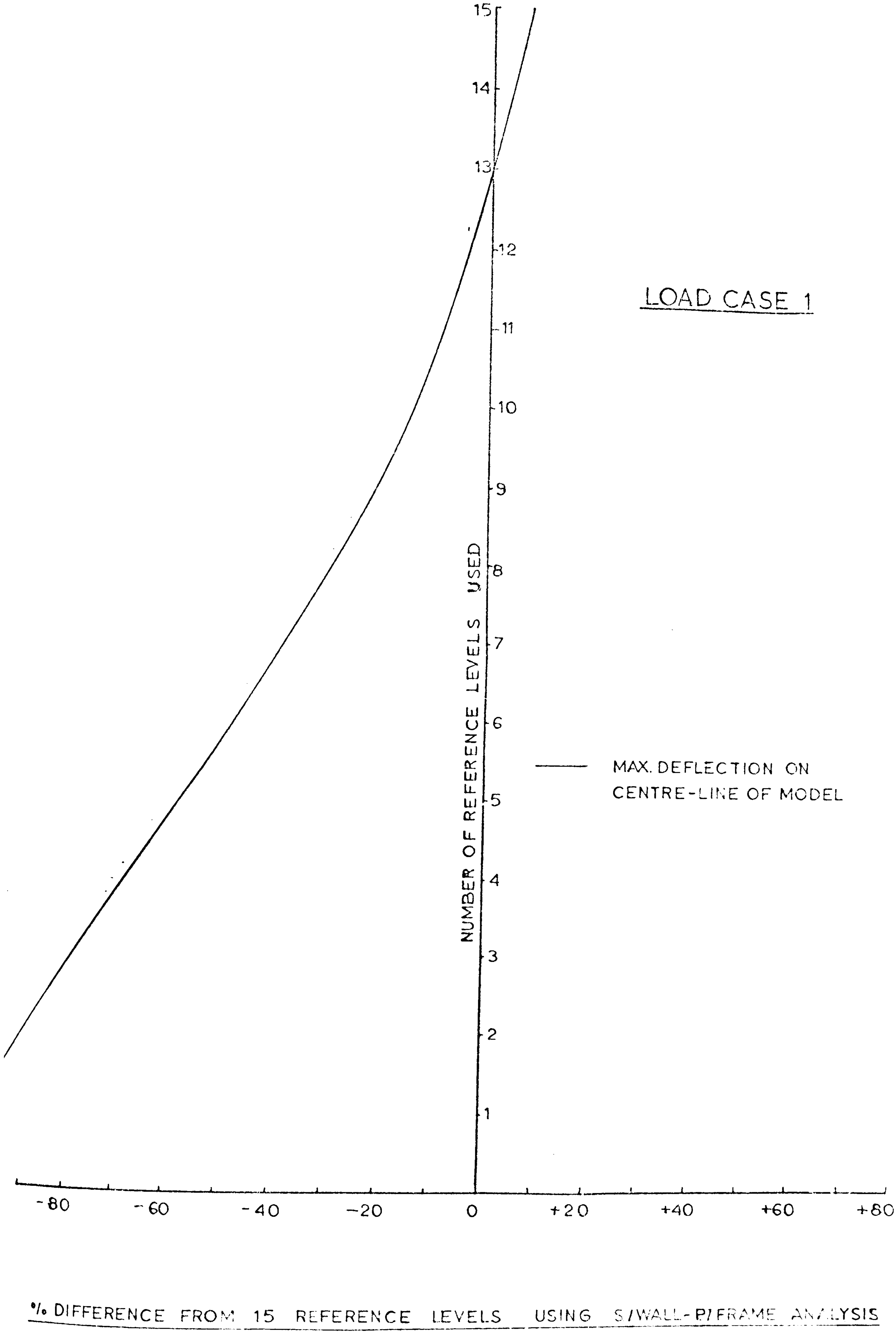

LOAD CASE 1
MAX. DEFLECTION ON CENTRE-LINE OF MODEL
NUMBER OF REFERENCE LEVELS USED
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
-80
-60
-40
-20
0
+20
+40
+60
+80
% DIFFERENCE FROM 15 REFERENCE LEVELS USING S/WALL-P/FRAME ANALYSIS

Fig. B-5.3

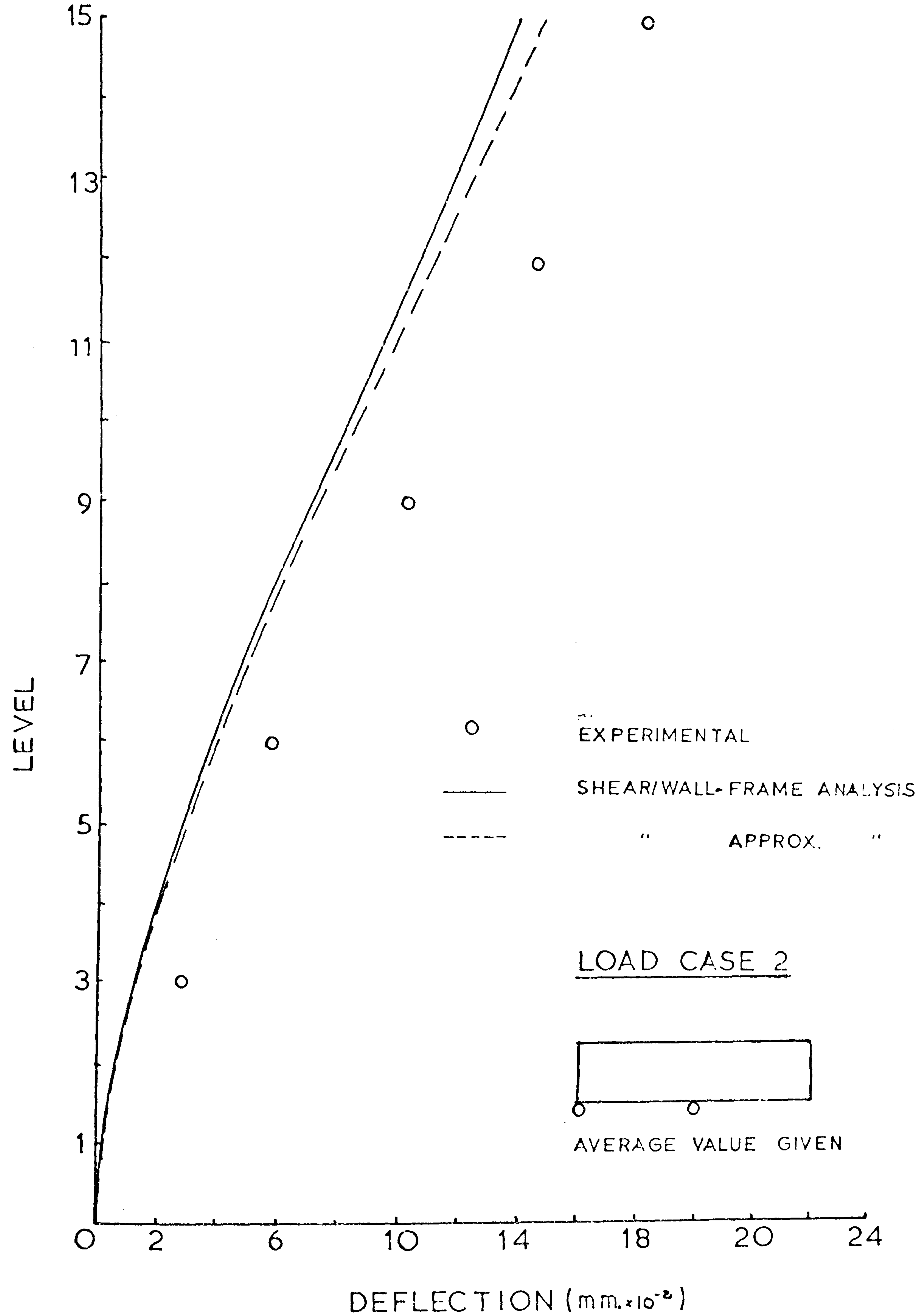

15
13
11
9
7
5
3
1
LEVEL
O
2
6
10
14
18
20
24
DEFLECTION (mm.×10⁻²)
EXPERIMENTAL
SHEAR/WALL-FRAME ANALYSIS
" APPROX. "
LOAD CASE 2
AVERAGE VALUE GIVEN

Fig. B-5.4

Fig.B-5.5

LOAD CASE 2

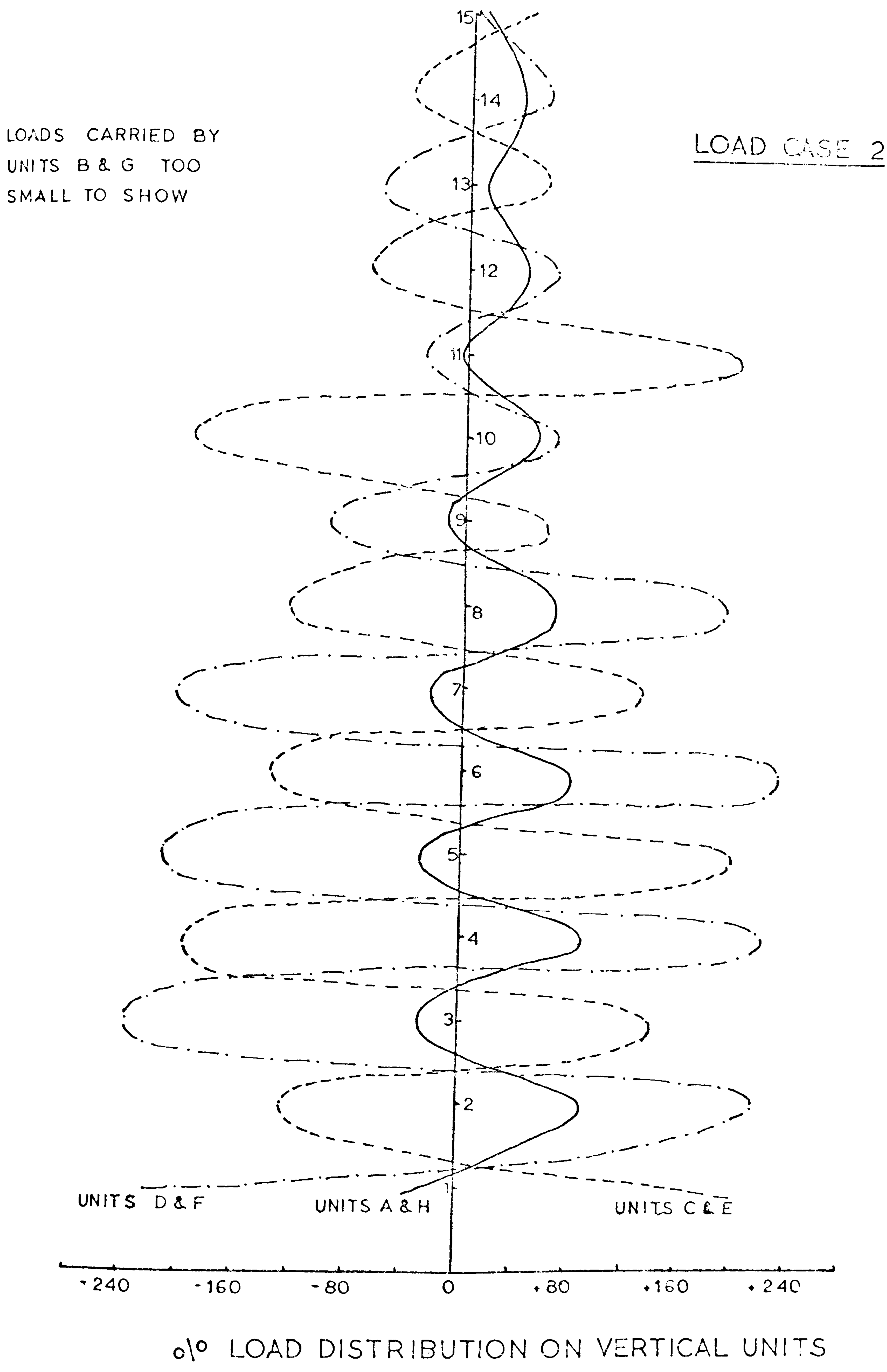
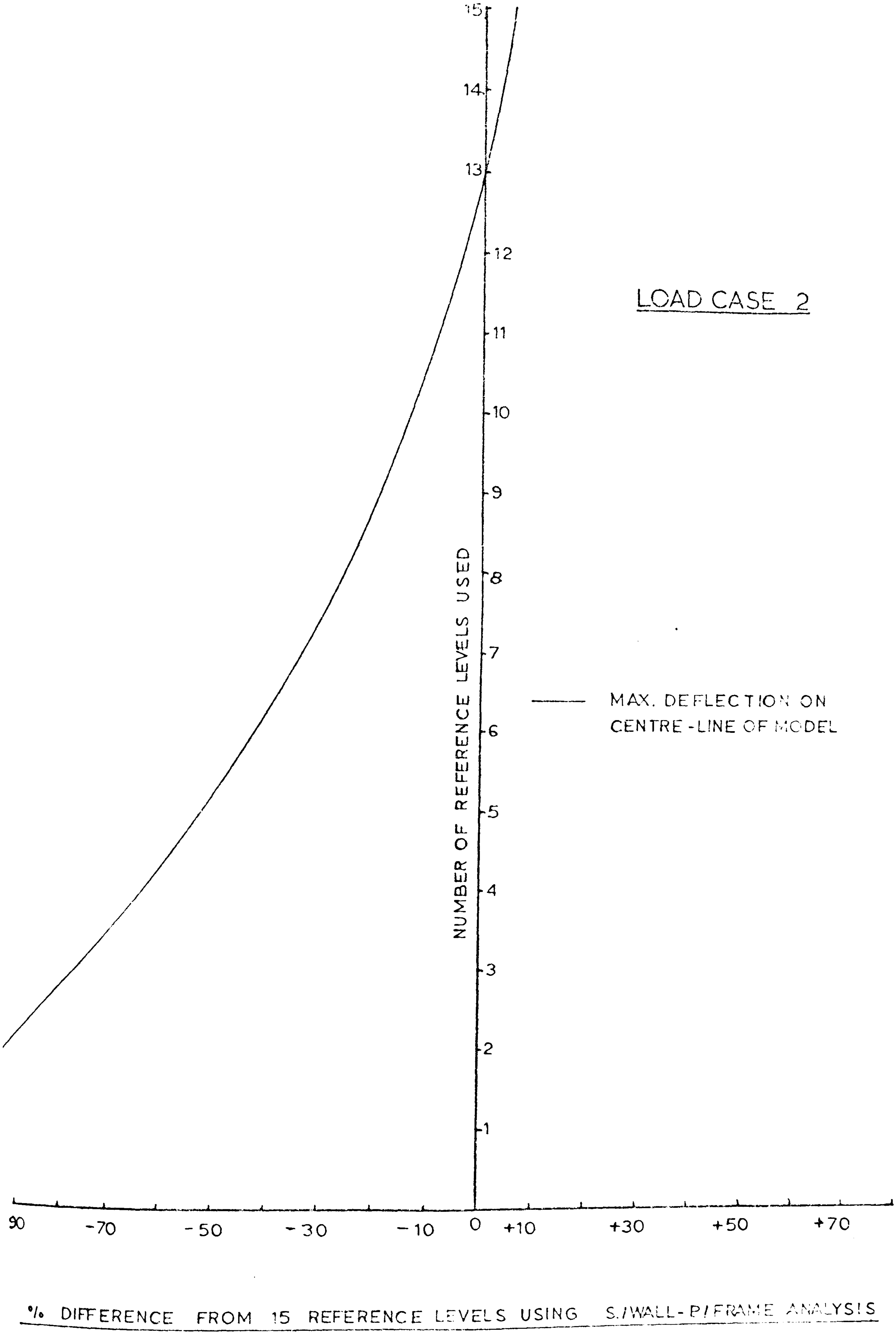
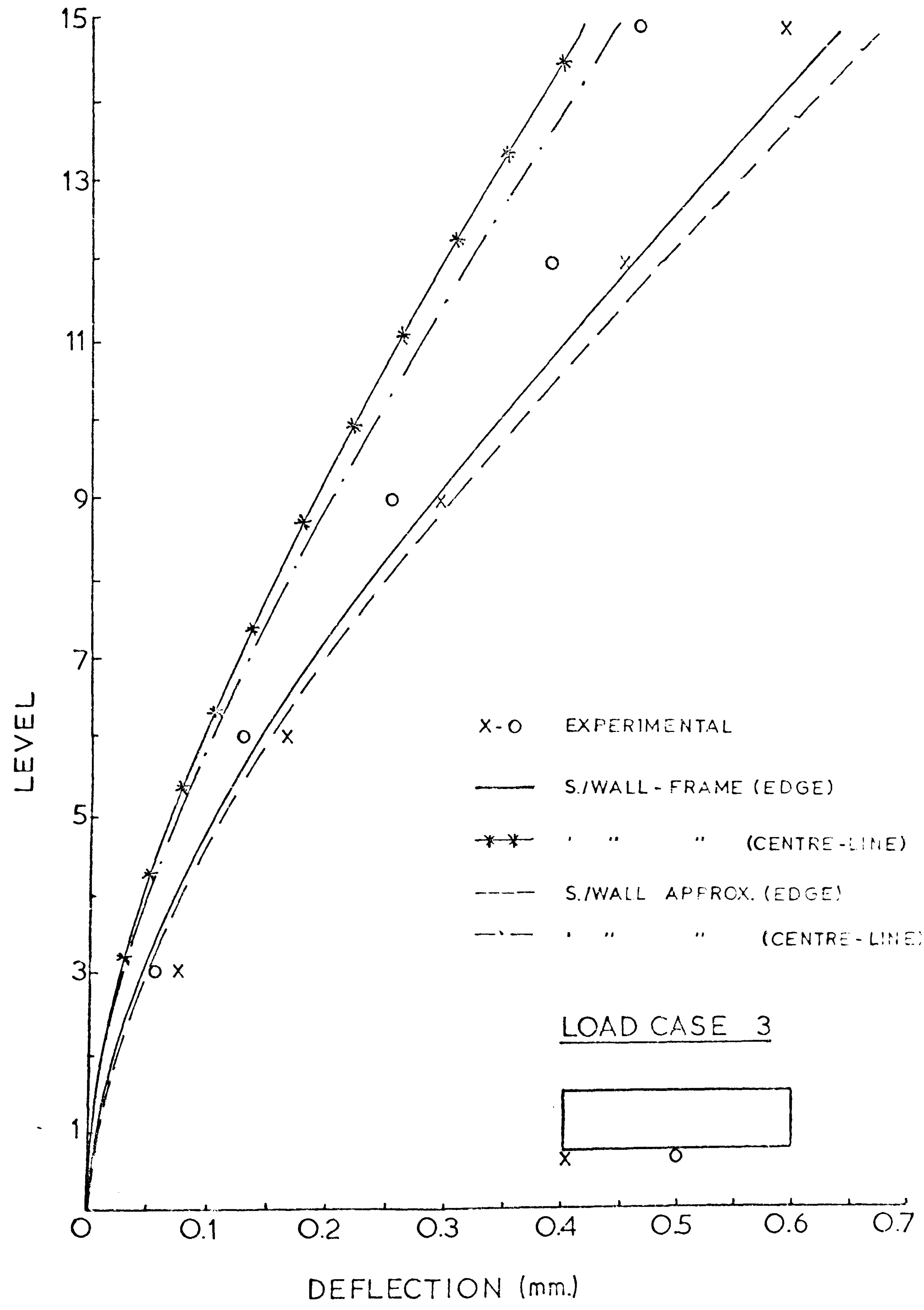

LOADS CARRIED BY
UNITS B & G TOO
SMALL TO SHOW
LOAD CASE 2
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
UNITS D & F
UNITS A & H
UNITS C & E
-240
-160
-80
0
+80
+160
+240
% LOAD DISTRIBUTION ON VERTICAL UNITS

Fig.B-5.6



LOAD CASE 2
MAX. DEFLECTION ON CENTRE-LINE OF MODEL
NUMBER OF REFERENCE LEVELS USED
% DIFFERENCE FROM 15 REFERENCE LEVELS USING S./WALL-P/FRAME ANALYSIS

Fig. B-5.7



15
13
11
9
7
5
3
1
LEVEL
O
0.1
0.2
0.3
0.4
0.5
0.6
0.7
DEFLECTION (mm.)
X-O EXPERIMENTAL
S./WALL-FRAME (EDGE)
" " " (CENTRE-LINE)
S./WALL APPROX. (EDGE)
" " " (CENTRE-LINE)
LOAD CASE 3
X
O

Fig.B-5.8

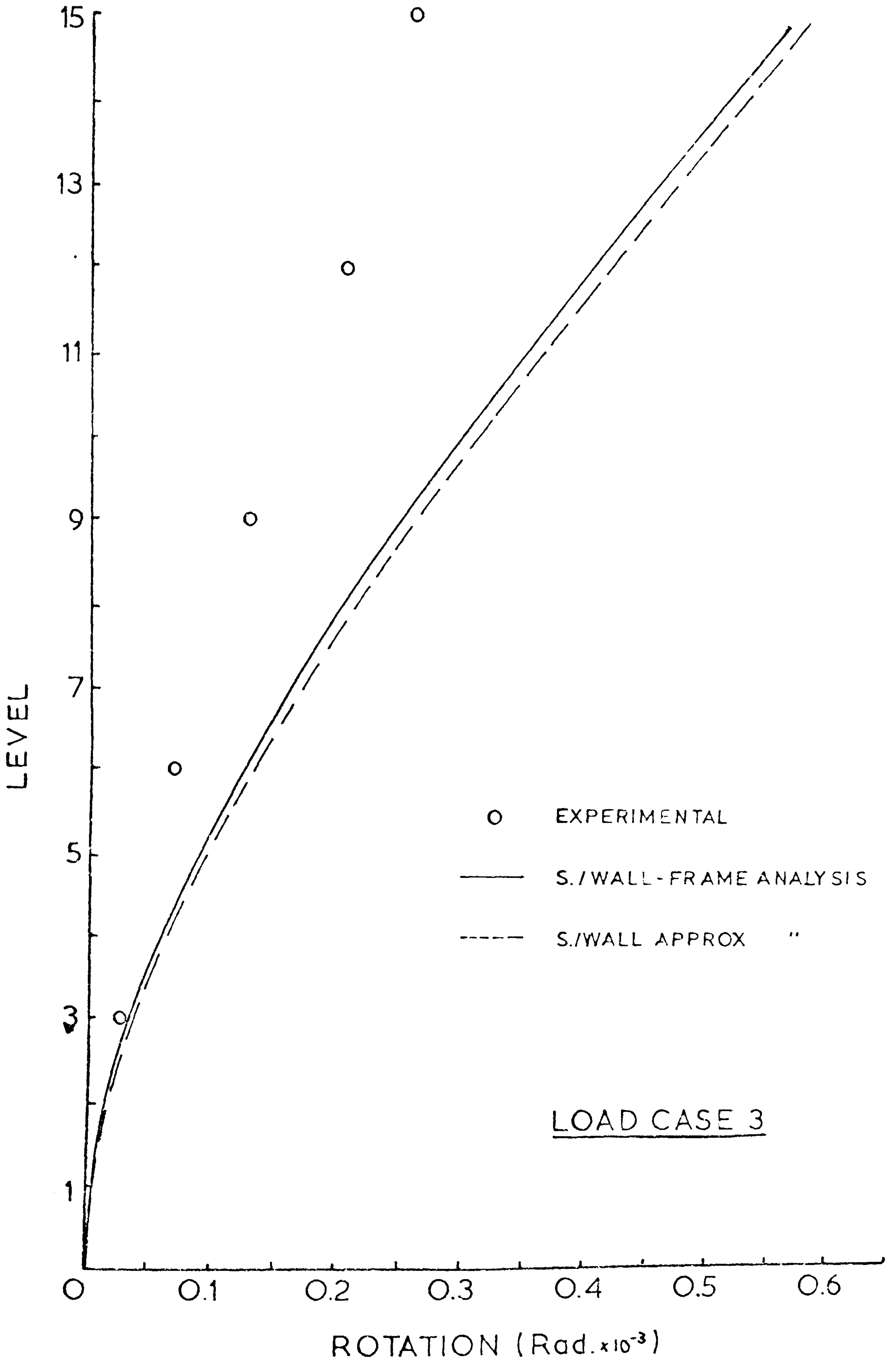

15
13
11
9
7
5
3
1
0
0.1
0.2
0.3
0.4
0.5
0.6
LEVEL
ROTATION (Rad.x10^-3)
EXPERIMENTAL
S./WALL-FRAME ANALYSIS
S./WALL APPROX "
LOAD CASE 3

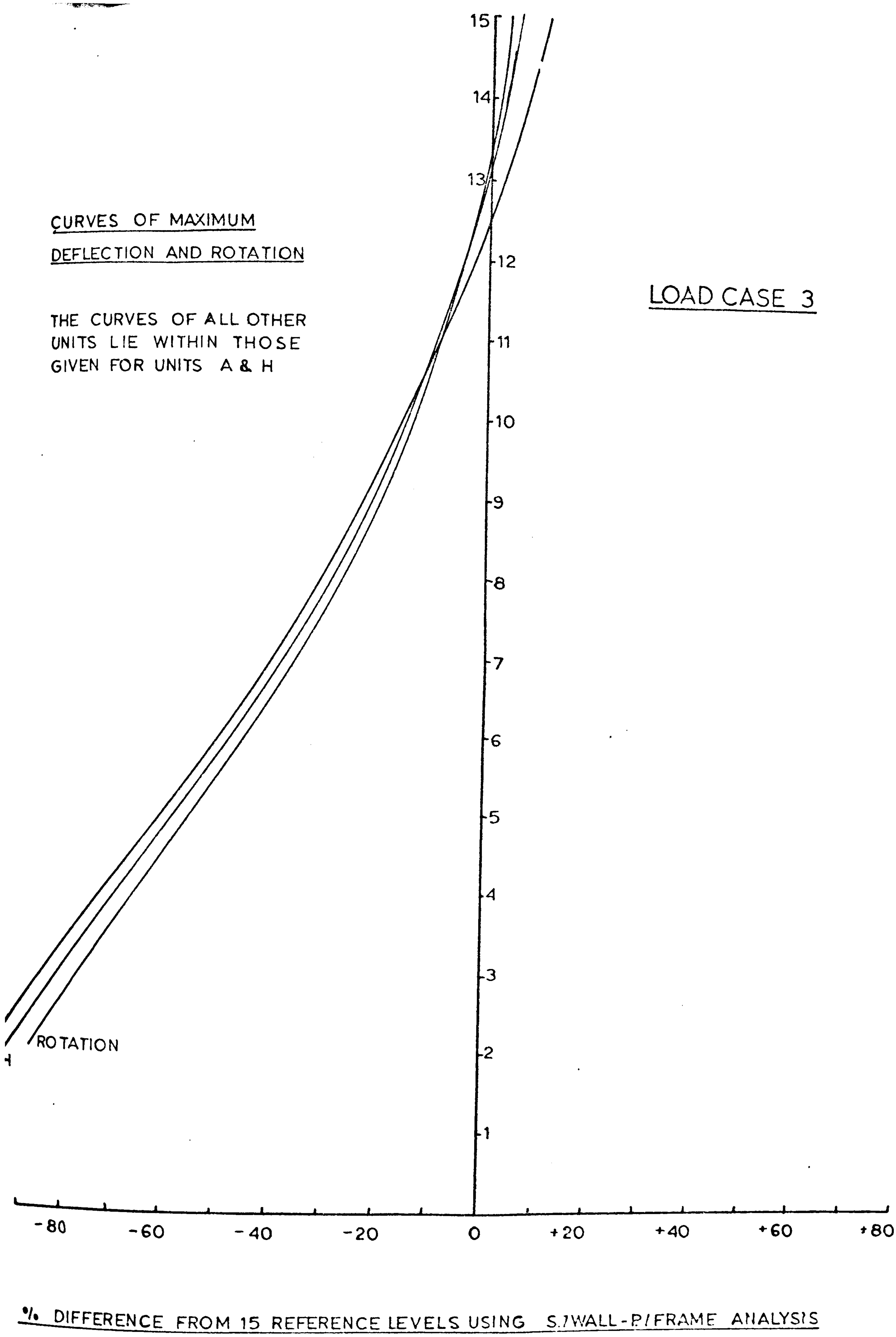
Fig.B-5.9

X	EXPERIMENTAL	LEVEL	2+10 mm.
O	"	"	1+10 mm.
—·—	THEORETICAL	"	2+10 mm.
——	"	"	1+10 mm.

Fig. B-5.10

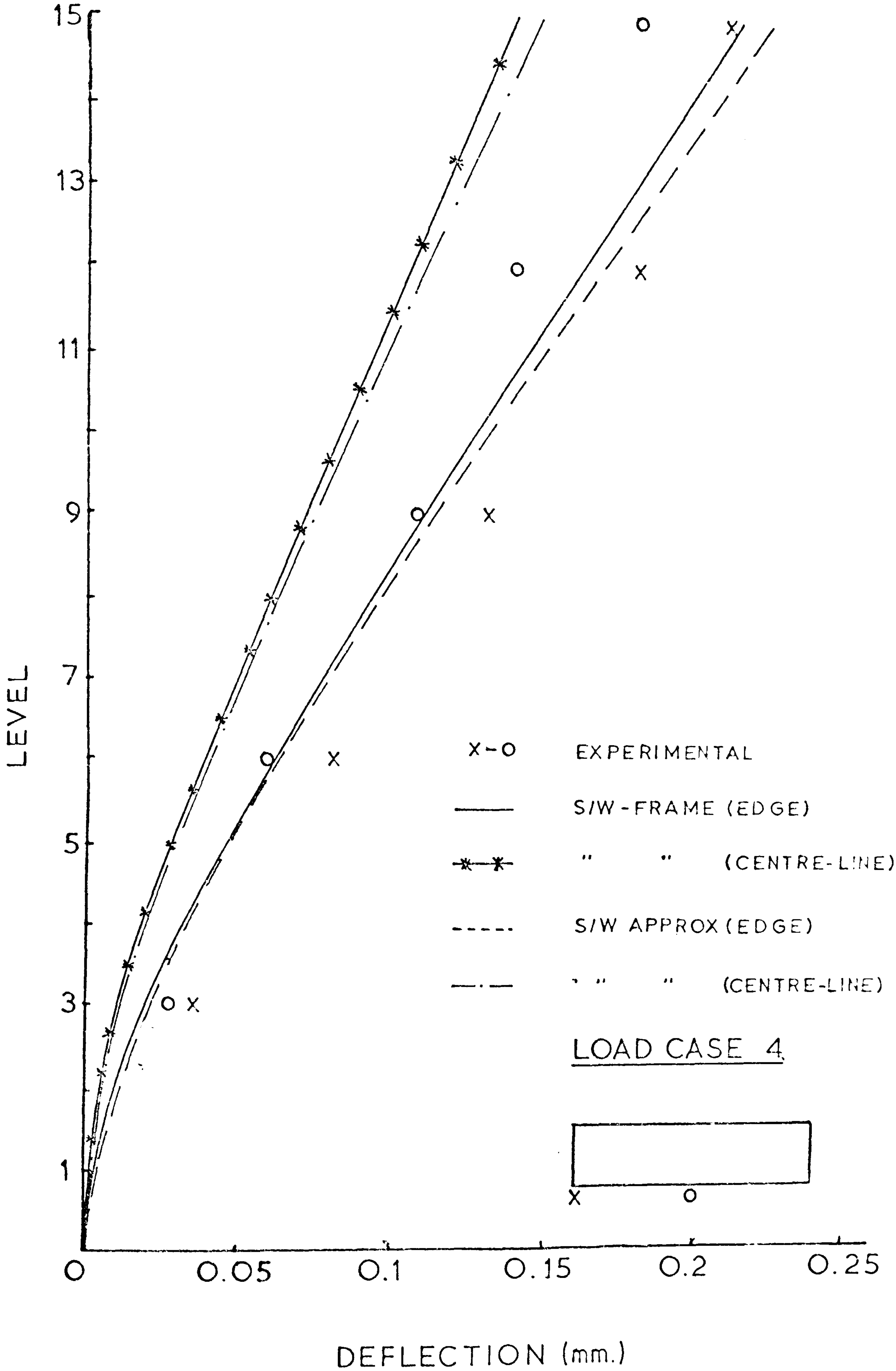
0 50 100 150
MICRO-STRAIN

LOAD CASE 3



CURVES OF MAXIMUM
DEFLECTION AND ROTATION
THE CURVES OF ALL OTHER
UNITS LIE WITHIN THOSE
GIVEN FOR UNITS A & H
LOAD CASE 3
ROTATION
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
-80
-60
-40
-20
0
+20
+40
+60
+80
% DIFFERENCE FROM 15 REFERENCE LEVELS USING S.7WALL-P/FRAME ANALYSIS

Fig. B-5.11



LEVEL
15
13
11
9
7
5
3
1
0
0.05
0.1
0.15
0.2
0.25
DEFLECTION (mm.)
X – O EXPERIMENTAL
S/W - FRAME (EDGE)
" " (CENTRE-LINE)
S/W APPROX (EDGE)
" " (CENTRE-LINE)
LOAD CASE 4
X
O

Fig. B-5.12

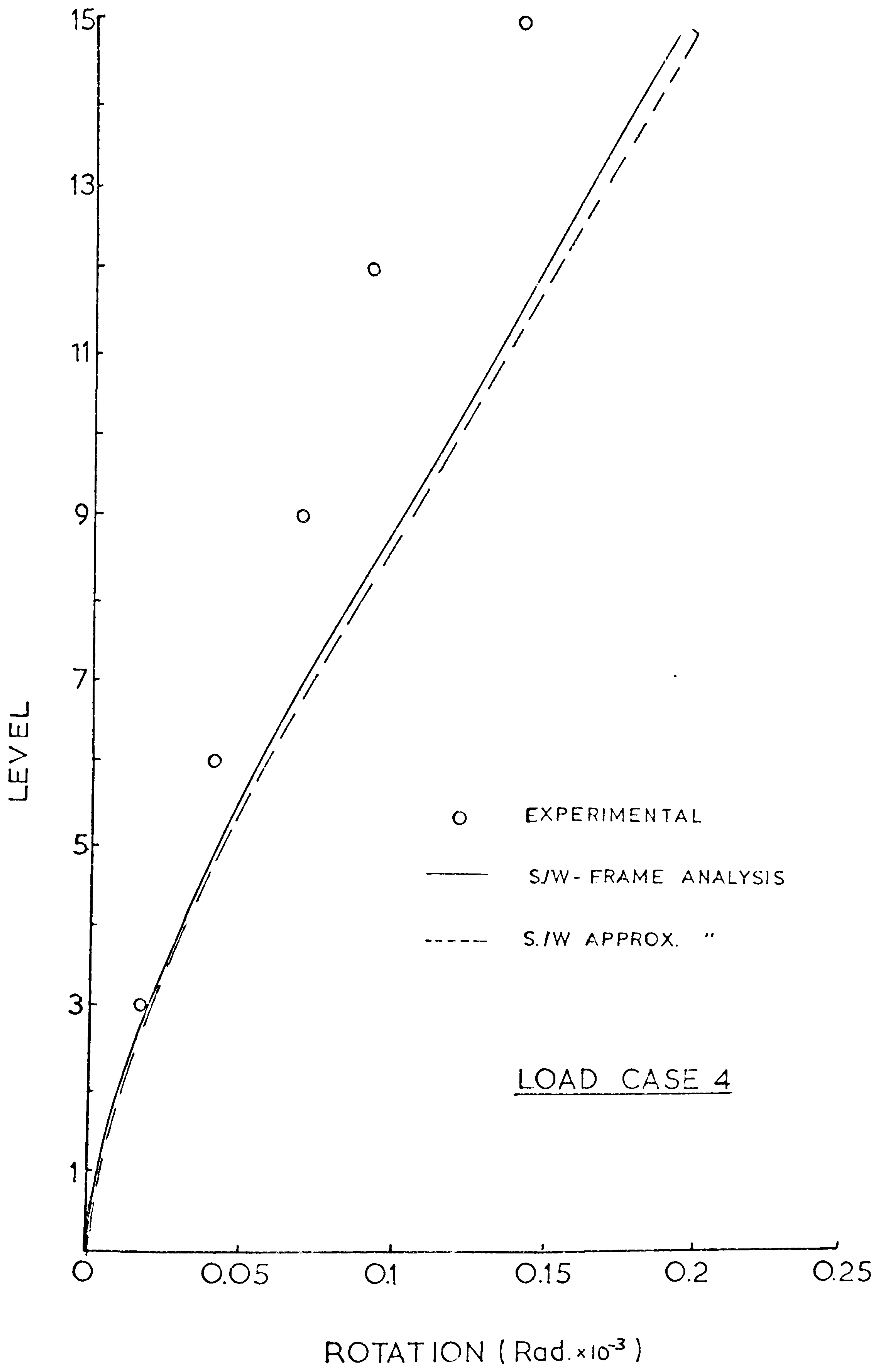

15
13
11
9
7
5
3
1
LEVEL
EXPERIMENTAL
S/W- FRAME ANALYSIS
S./W APPROX. "
LOAD CASE 4
0
0.05
0.1
0.15
0.2
0.25
ROTATION (Rad. x10^{-3})

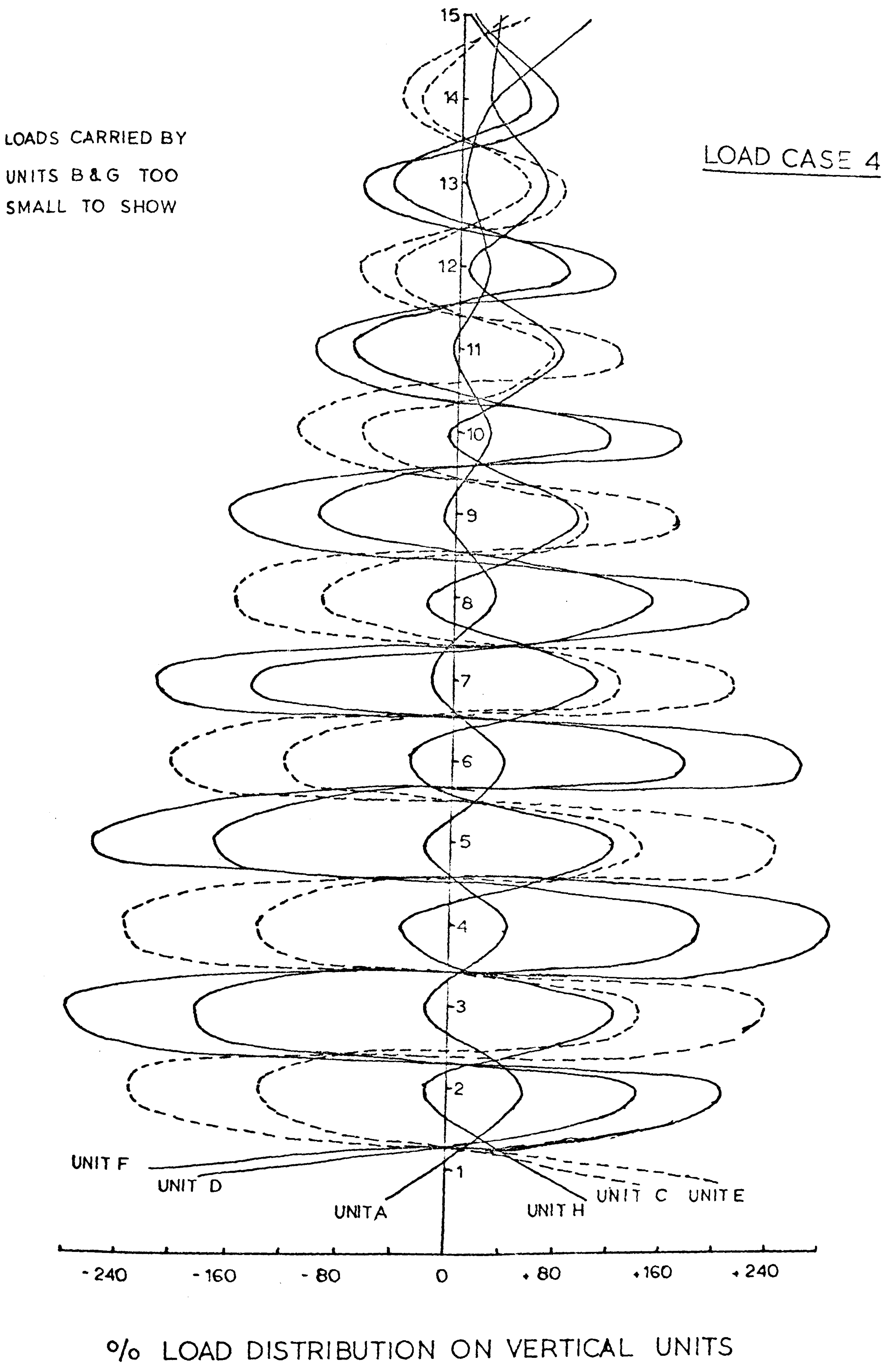
Fig. B-5.13

x EXPERIMENTAL LEVEL 2+10mm.
o " " 1+10mm.
—·— THEORETICAL " 2+10mm.
—— " " 1+10mm.

0 25 50 75
MICRO-STRAIN

Fig.B-5.14

LOAD CASE 4



LOADS CARRIED BY
UNITS B & G TOO
SMALL TO SHOW
LOAD CASE 4
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
UNIT F
UNIT D
UNIT A
UNIT H
UNIT C
UNIT E
-240
-160
-80
0
+80
+160
+240
% LOAD DISTRIBUTION ON VERTICAL UNITS

Fig. B-5.15

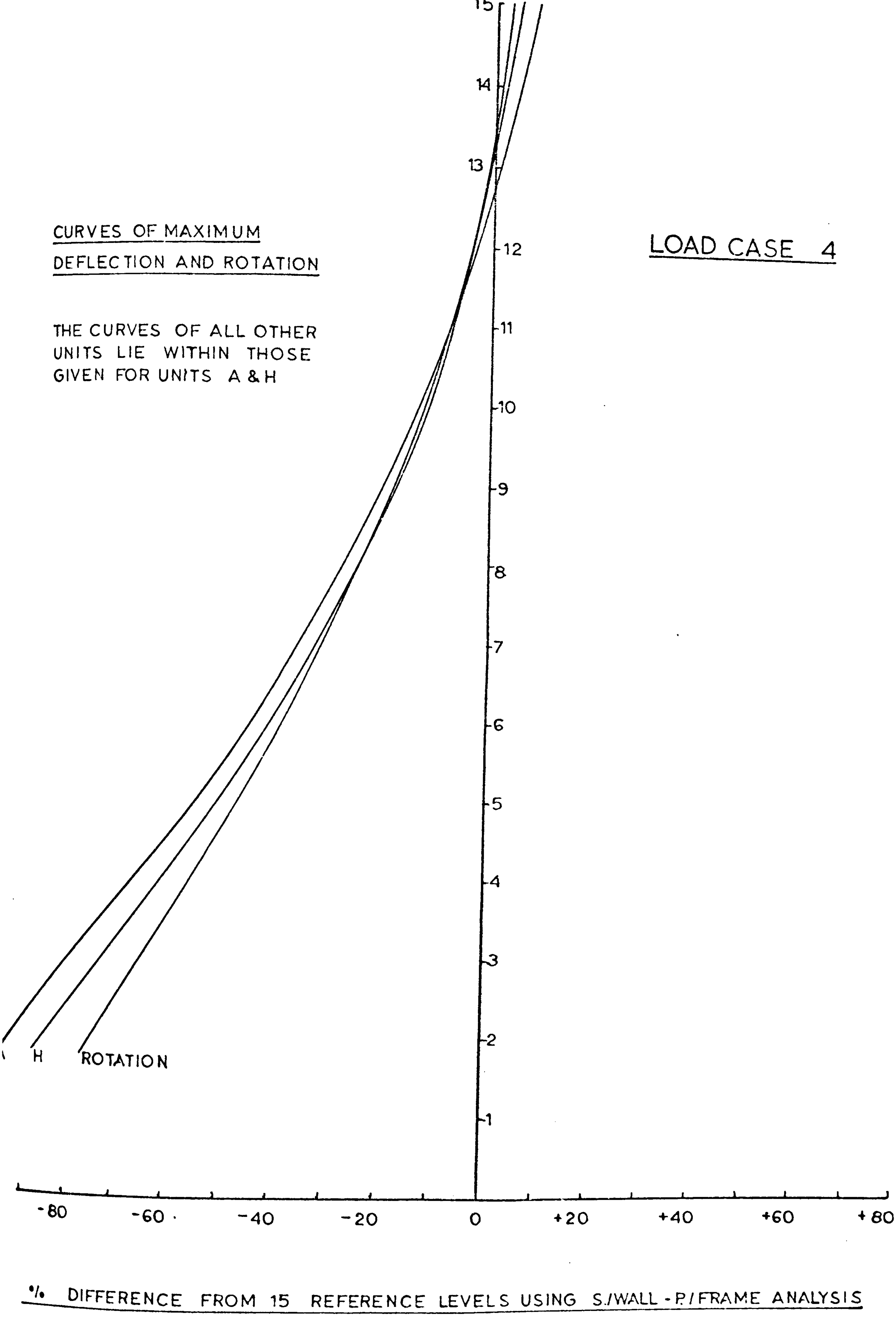
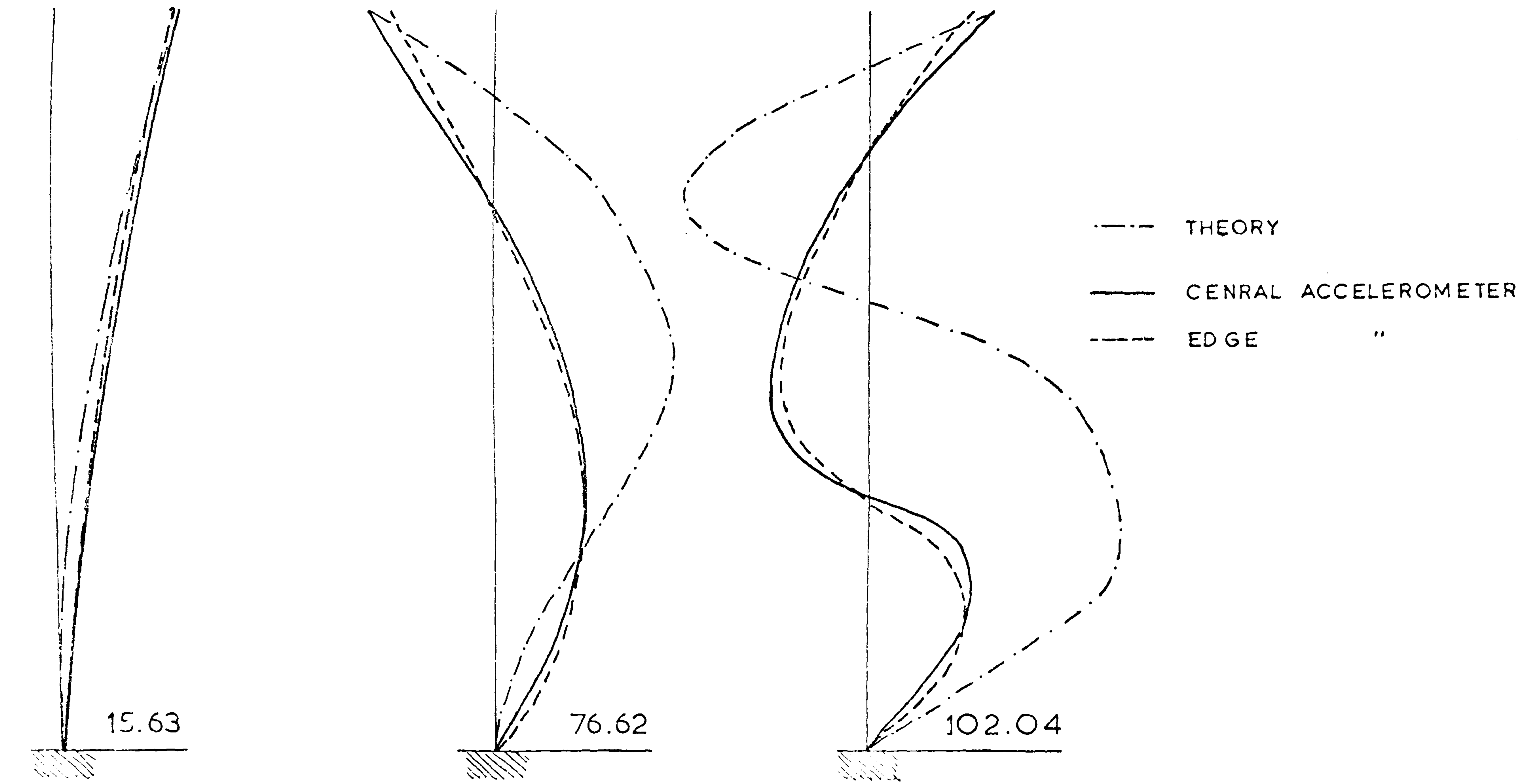

CURVES OF MAXIMUM
DEFLECTION AND ROTATION
THE CURVES OF ALL OTHER
UNITS LIE WITHIN THOSE
GIVEN FOR UNITS A & H
LOAD CASE 4
H
ROTATION
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
-80
-60
-40
-20
0
+20
+40
+60
+80
% DIFFERENCE FROM 15 REFERENCE LEVELS USING S./WALL - P./FRAME ANALYSIS

Fig.B -5.16

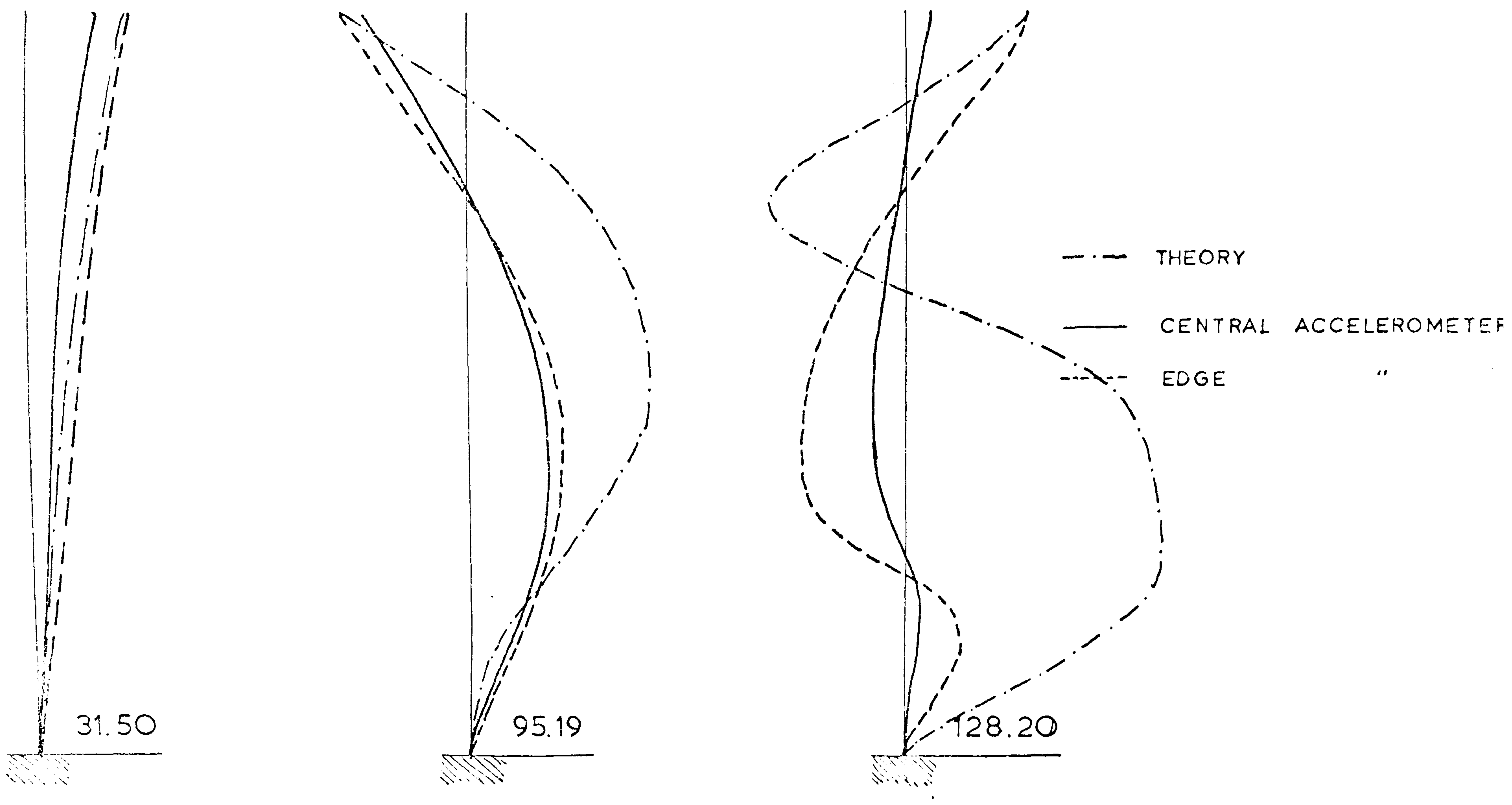
CENTRAL ACCELEROMETER LEVEL 15 PARALLEL TO VERTICAL UNITS
EDGE " " " " " " "
1ST Bending
1ST Torsion
2nd Bending
Test Rig
2nd Torsion
3rd Bending
3rd Torsion
0 10 20 30 40 50 60 70 80 90 100 110 120 130
FREQUENCY (Hz.)
A-AMPLITUDE
MODEL No. 5 DYNAMIC RESPONSE CURVE

Fig. B-5.18



TRANSTLATIONAL — MODE SHAPES AND NATURAL FREQ. (Hz)

Fig. B-5.19



ROTATIONAL— MODE SHAPES AND NATURAL FREQ.(Hz)

Fig.B-5.20

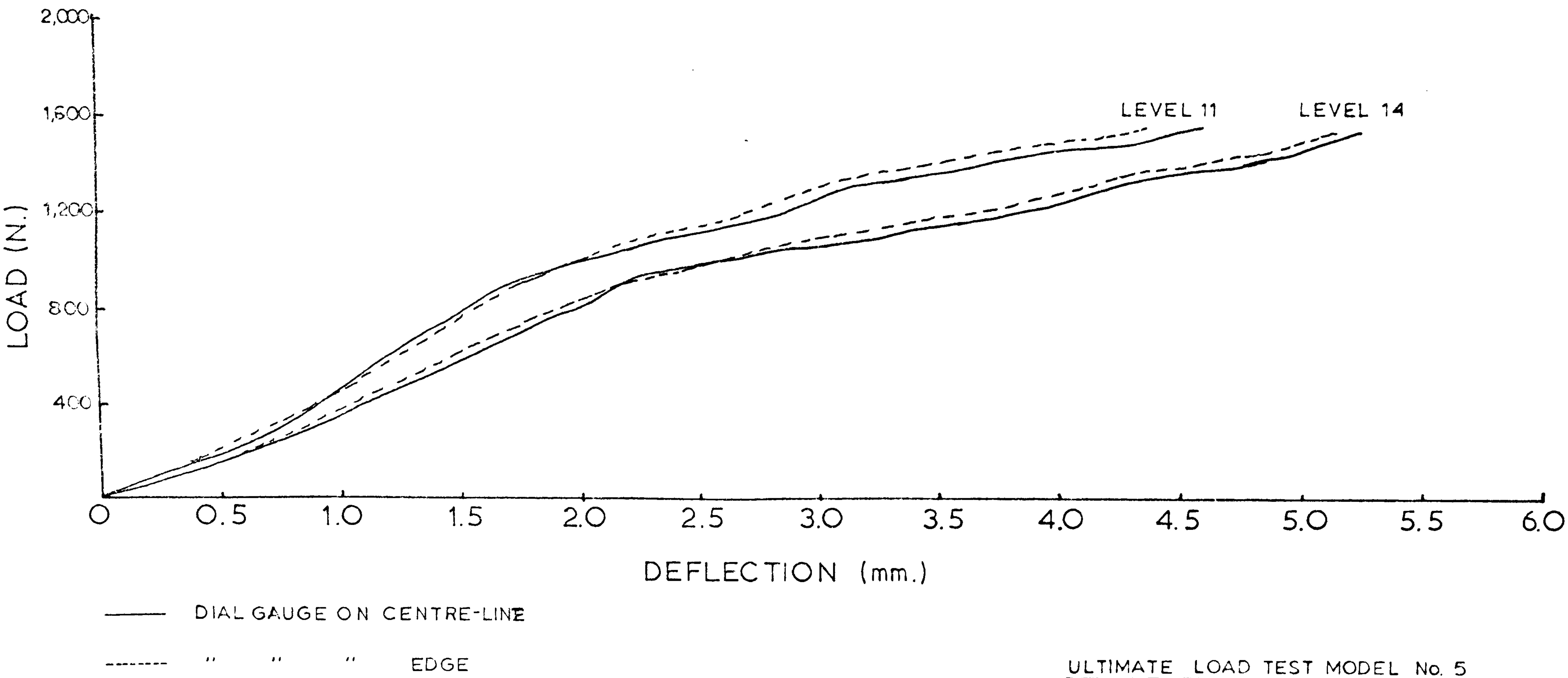

EXPERIMENT
THEORY
MODE
3
2
1
TRANSLATION
ROTATION
20
40
60
80
100
120
140
160
180
200
220
240
FREQUENCY (Hz.)

Fig. B-21

NATURAL FREQ.(Hz.)		MODE TYPE	% CRITICAL DAMPING	
Experiment	Theory		Forced Vibrations	Free Vibrations
15.63	20.30	Translation	3.73	5.46
31.50	17.43	Rotation	2.21	
76.62	102.42	Translation	2.88	
95.19	84.72	Rotation	6.85	
102.04	249.97	Translation	1.83	
128.20	204.69	Rotation	0.69	

NATURAL FREQUENCIES, MODE TYPES AND DAMPING VALUES FOR MODEL 5

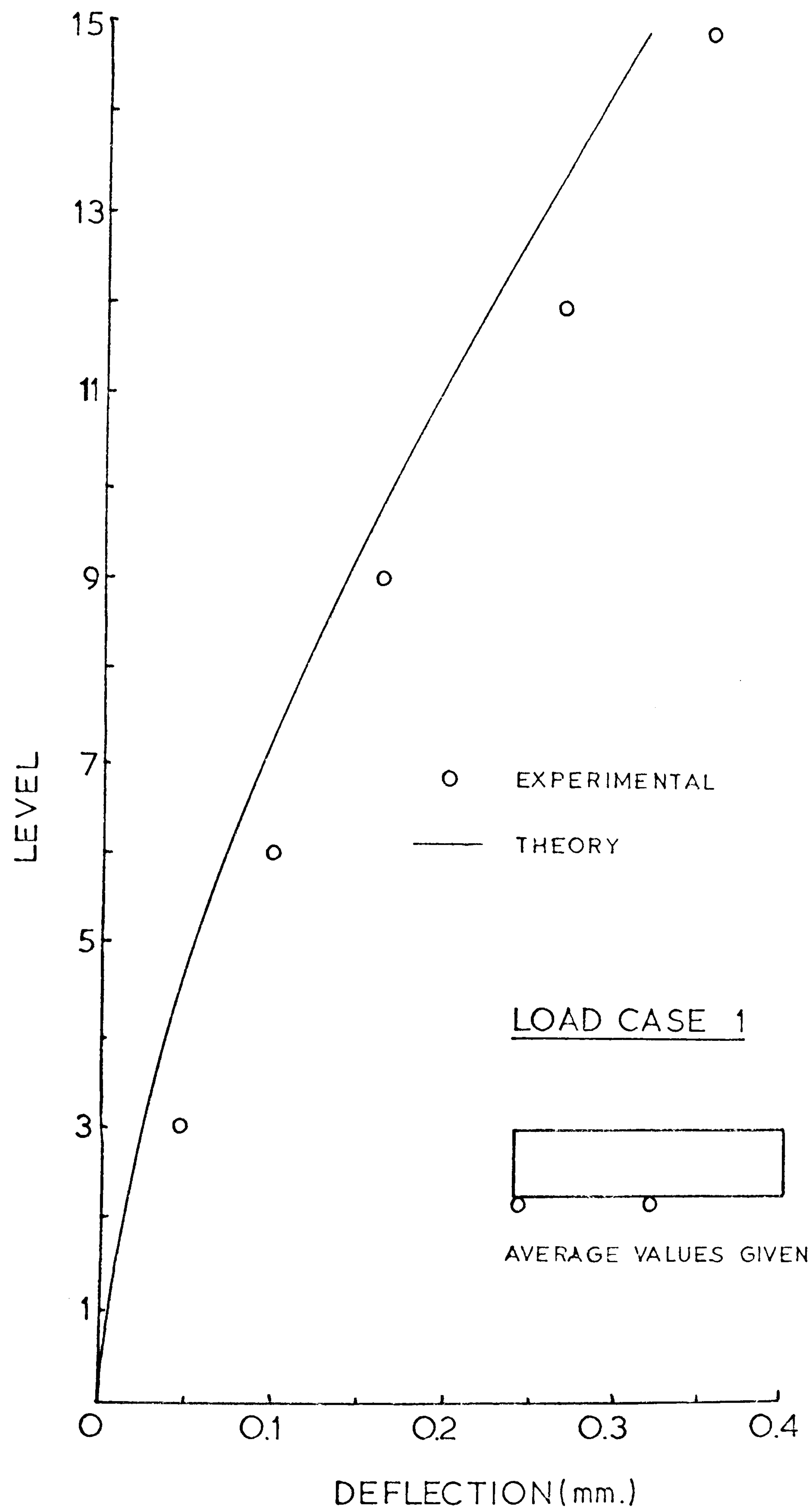
Table B-5



ULTIMATE LOAD TEST MODEL No. 5

Fig.B-5.22

Model No.6



15
13
11
9
7
5
3
1
LEVEL
0
0.1
0.2
0.3
0.4
DEFLECTION (mm.)
EXPERIMENTAL
THEORY
LOAD CASE 1
AVERAGE VALUES GIVEN

Fig B-6.1

x	EXPERIMENTAL	LEVEL	2+10mm.
o	"	"	1+10mm.
—·—	THEORETICAL	"	2+10mm.
——	"	"	1+10mm.

Fig. B-6.2

0 50 100 150

MICRO-STRAIN

LOAD CASE 1

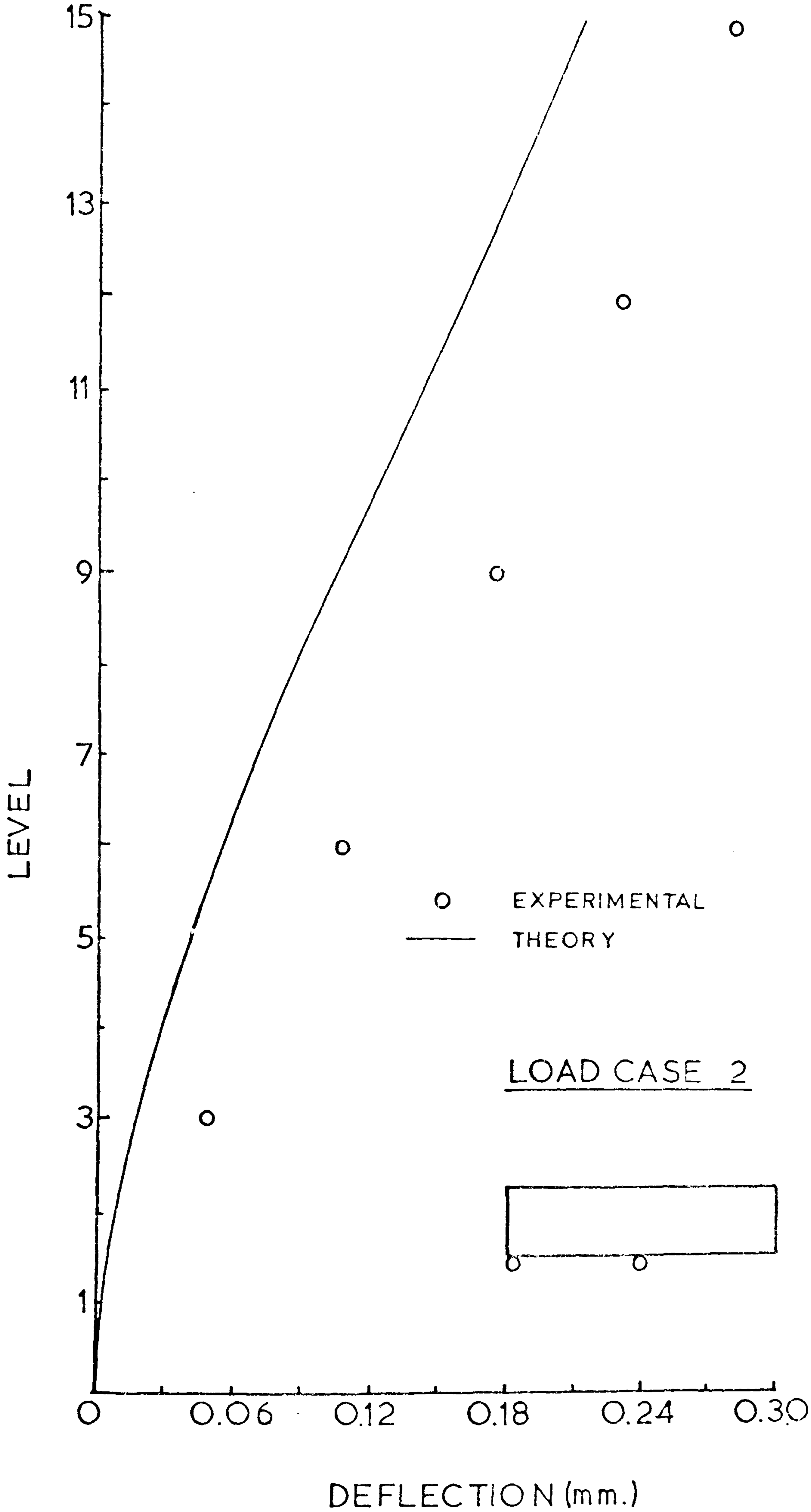

15
13
11
9
7
5
3
1
LEVEL
O
0.06
0.12
0.18
0.24
0.30
DEFLECTION (mm.)
EXPERIMENTAL
THEORY
LOAD CASE 2

Fig. B-6.3

x	EXPERIMENTAL	LEVEL	2 + 10 mm.
o	"	"	1 + 10 mm.
— · —	THEORETICAL	"	2 + 10mm.
——	"	"	1 + 10 mm.

0 25 50 75

MICRO - STRAIN

Fig. B-6.4

LOAD CASE 2

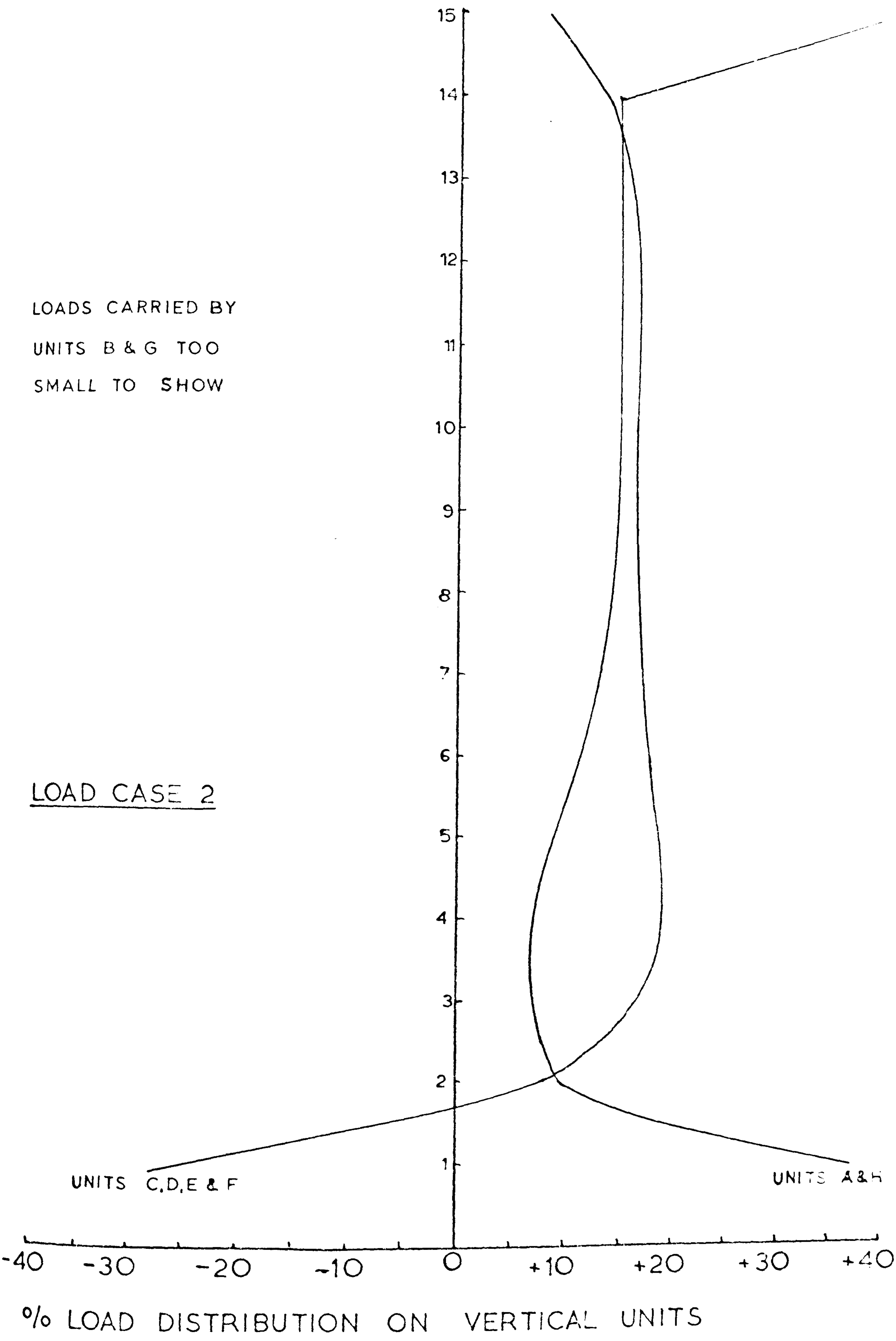

LOADS CARRIED BY
UNITS B & G TOO
SMALL TO SHOW
LOAD CASE 2
UNITS C,D,E & F
UNITS A&H
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
-40
-30
-20
-10
0
+10
+20
+30
+40
% LOAD DISTRIBUTION ON VERTICAL UNITS

Fig. B-6.5

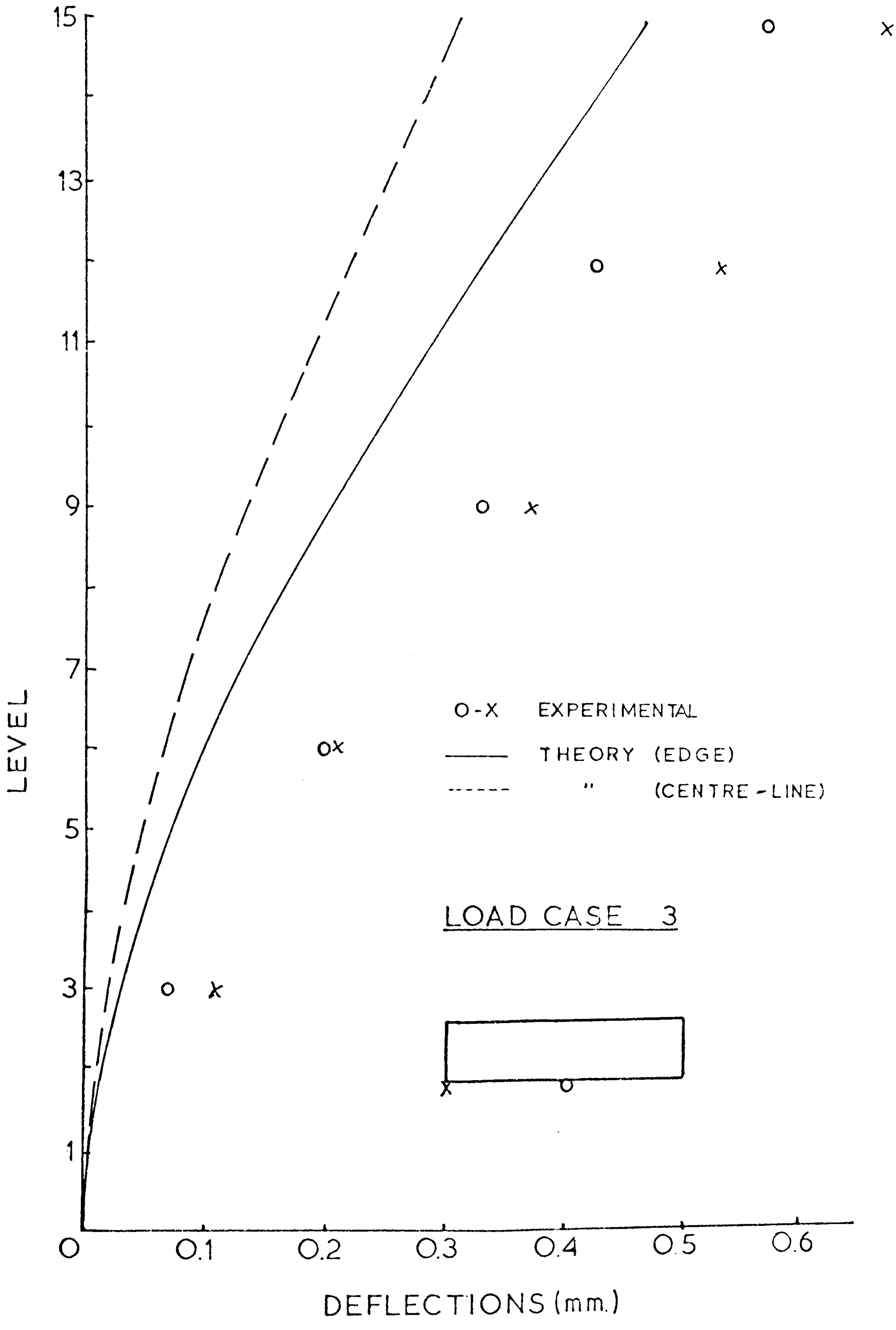
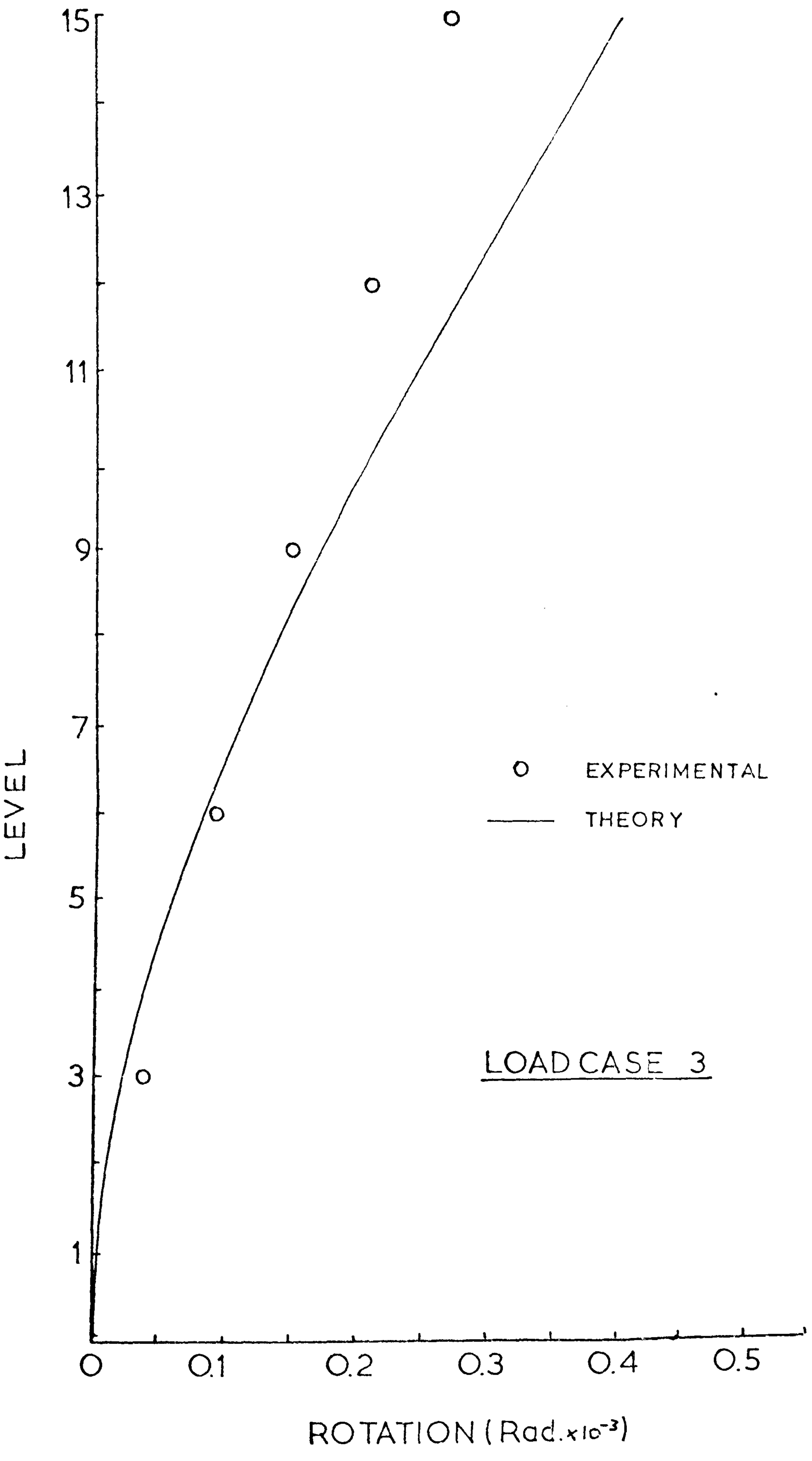

15
13
11
9
7
5
3
1
LEVEL
O
0.1
0.2
0.3
0.4
0.5
0.6
DEFLECTIONS (mm.)
O-X EXPERIMENTAL
THEORY (EDGE)
" (CENTRE-LINE)
LOAD CASE 3

Fig.B-6.6

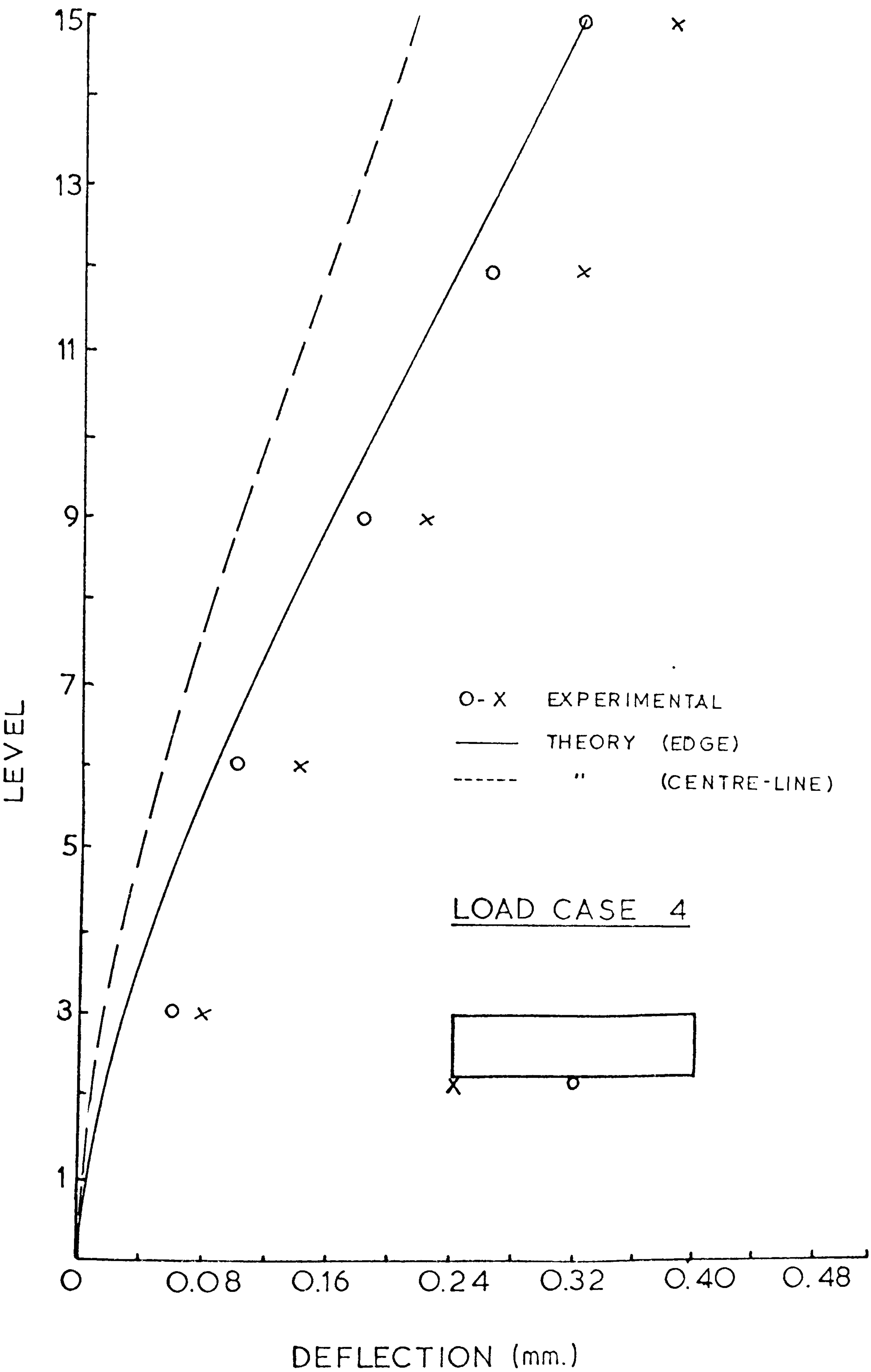


15
13
11
9
7
5
3
1
LEVEL
O EXPERIMENTAL
THEORY
LOAD CASE 3
0
0.1
0.2
0.3
0.4
0.5
ROTATION (Rad. x10^-3)

Fig.B-6.7

X EXPERIMENTAL LEVEL 2+10mm.
O " " 1+10mm.
THEORETICAL " 2+10mm.
" " 1+10mm.
0 50 100 150
MICRO-STRAIN
LOAD CASE 3

Fig. B-6.8



15
13
11
9
7
5
3
1
LEVEL
O
0.08
0.16
0.24
0.32
0.40
0.48
DEFLECTION (mm.)
O-X EXPERIMENTAL
THEORY (EDGE)
" (CENTRE-LINE)
LOAD CASE 4

Fig. B-6.9

Fig.B-6.11

LOAD CASE 4

APPENDIX E

Photographs

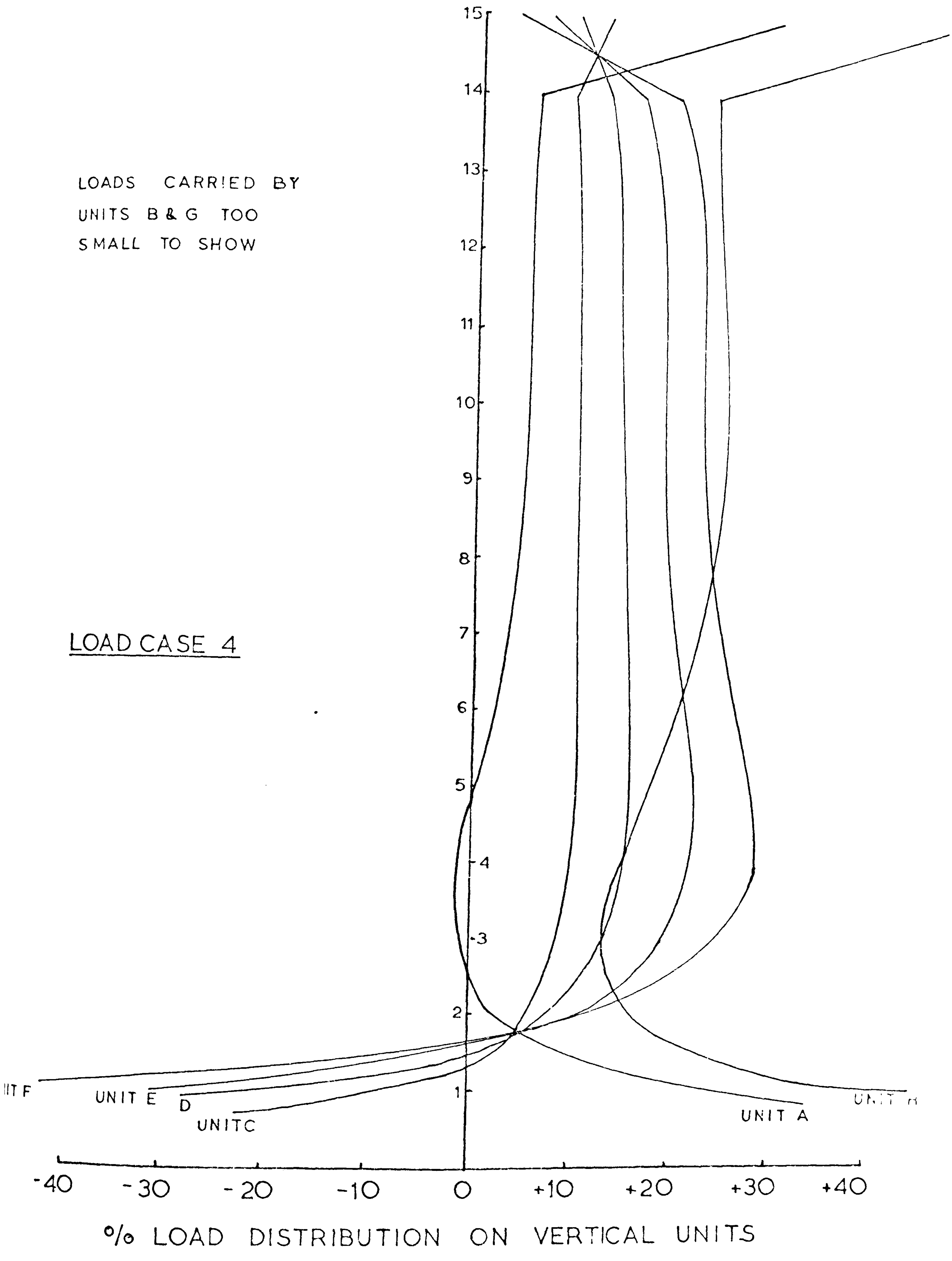
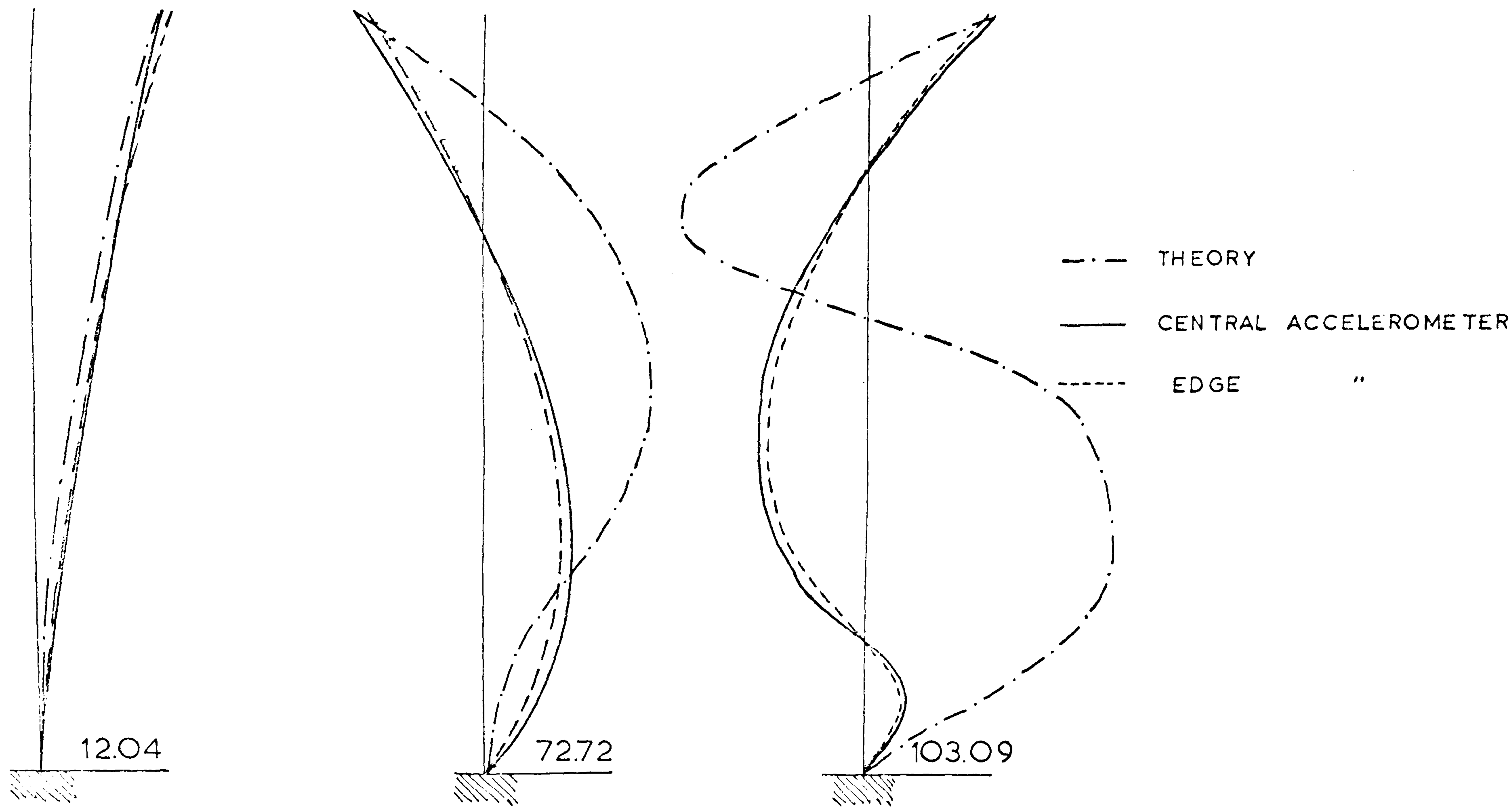

LOADS CARRIED BY
UNITS B & G TOO
SMALL TO SHOW
LOAD CASE 4
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
UNIT E
D
UNIT C
UNIT A
-40
-30
-20
-10
0
+10
+20
+30
+40
% LOAD DISTRIBUTION ON VERTICAL UNITS

Fig.B-6.12

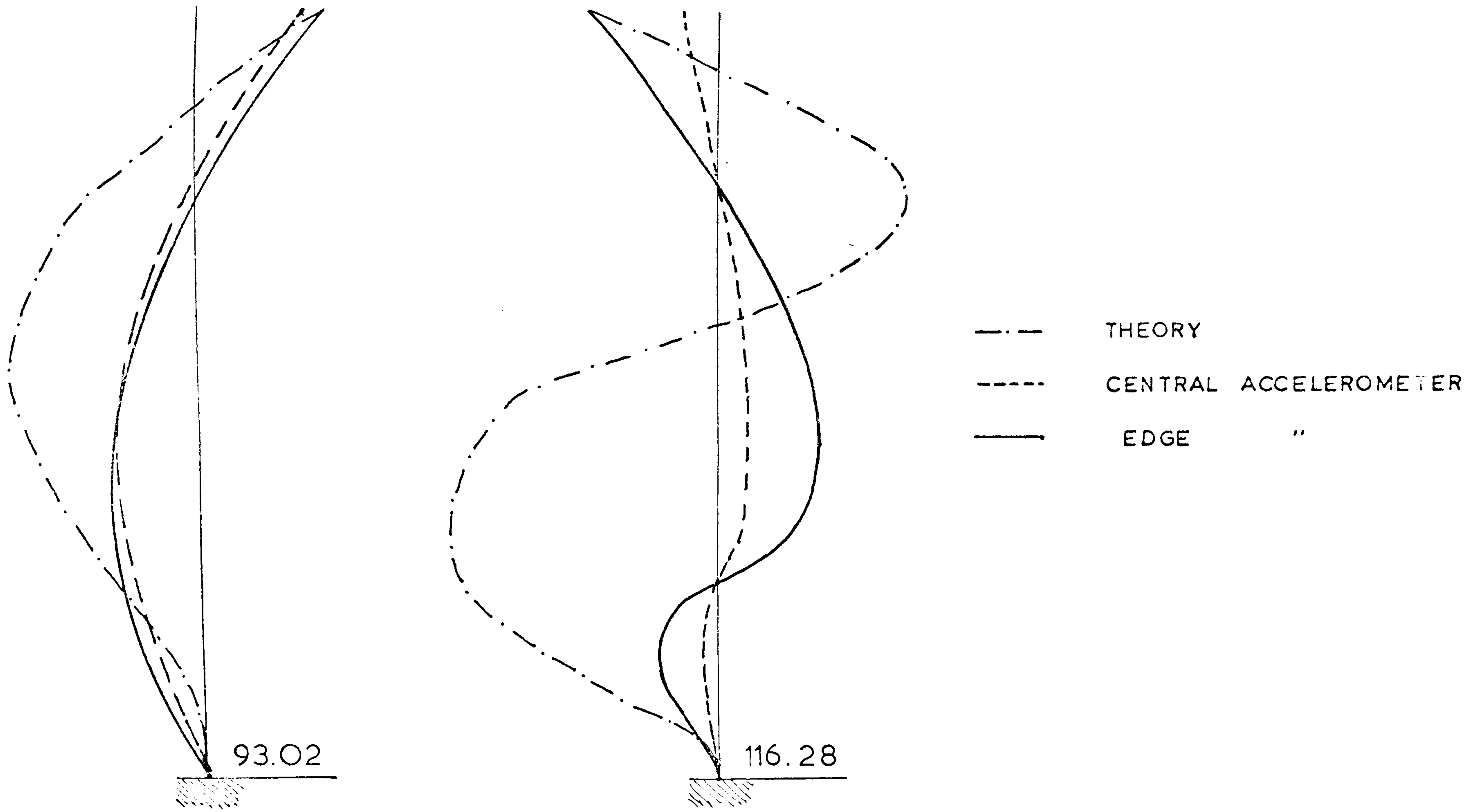
MODEL No. 6 DYNAMIC RESPONSE CURVE

Fig. B-6.13



TRANSLATIONAL —— MODE SHAPES AND NATURAL FREQ. (Hz.)

Fig.B-6.14



ROTATIONAL MODE SHAPES AND NATURAL FREQ.(Hz.)

Fig.B-6.15

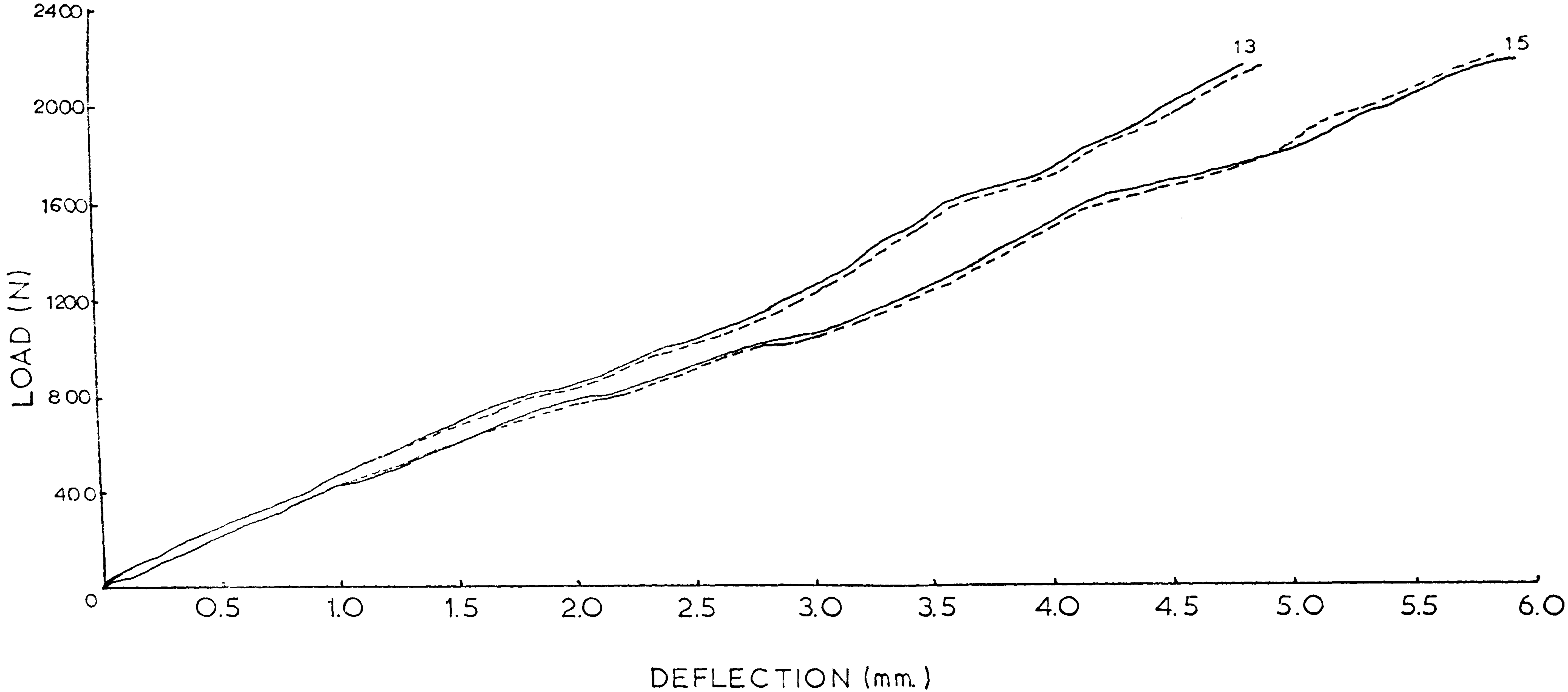
EXPERIMENT
THEORY
MODE
3rd
2nd
1st
TRANSLATION
ROTATION
20
40
60
80
100
120
140
160
180
200
220
240
FREQUENCY (Hz.)

Fig. B-6.16

NATURAL FREQ. (Hz.)		MODE TYPE	% CRITICAL DAMPING	
Experiment	Theory		Forced Vibrations	Free Vibrations
12.04	24.54	Translation	2.88	6.02
—	18.95	Rotation	—	
72.72	119.79	Translation	2.20	
93.02	93.49	Rotation	0.52	
103.09	290.77	Translation	1.71	
116.28	227.80	Rotation	2.39	

NATURAL FREQUENCIES, MODE TYPES AND DAMPING VALUES FOR MODEL 6

Table B-6



——— DIAL GAUGE ON CENTRE - LINE
- - - - " " " EDGE

ULTIMATE LOAD TEST MODEL No. 6

Fig.B-6.17

APPENDIX C

Given below is a brief description of the procedure used by Derecho in his computer analysis (21) to account for the interaction between adjacent frames in a staggered-wall-beam structure under lateral loading.

Two typical adjacent frame bents are shown in Fig. C-1(a and b), the frames are shown lying in the same plane instead of parallel planes. The procedure is illustrated here for the simplest case, in which the columns are assumed to be very flexible so that the horizontal shear at any floor level is resisted entirely by the wall-beam at that level.

By assuming that all the horizontal shear is taken up by the wall-beams, the interacting forces at each floor level may be readily obtained by alternately transferring the total horizontal shear at any particular level to the frame with the wall-beam. Here, we are essentially assuming that the columns are hinged at their junctions with the wall-beams. Thus, starting from the beam marked "a" at the top of Frame Type A on/

on the left side of Fig. C-2, the horizontal shear at the bottom of the wall beam equals $(F_1 + F_2)$, which must be transferred to the top of the wall-beam marked "b" of the adjoining Frame Type B on the right side, since the columns below beam "a" cannot take this shear. If we next consider Frame Type B on the right side, we see that the force $(F_1 + F_2)$ transferred to the top of beam "b" now has to be transferred back to the top of the beam marked "c" in Frame Type A on the left side, since the columns below beam "b" also cannot take this shear. If this process is continued moving from top to bottom the "flow" of the total horizontal shear from one frame to the other results in the distribution shown in Fig. C-3. This method was used to determine the load distribution for Models 2, 3 and 4.

For the more general case where the columns are stiff enough relative to the wall-beams to take a significant proportion of the total horizontal shear, the interaction forces between two adjacent frames are determined in a similar manner to that described above. The portion of the horizontal shear at each floor level carried by the columns/

columns is determined approximately by considering the ratio of their lateral stiffness relative to that of the wall-beam of the adjoining frame at the same level. The procedure for this is fully explained in Ref. (21).

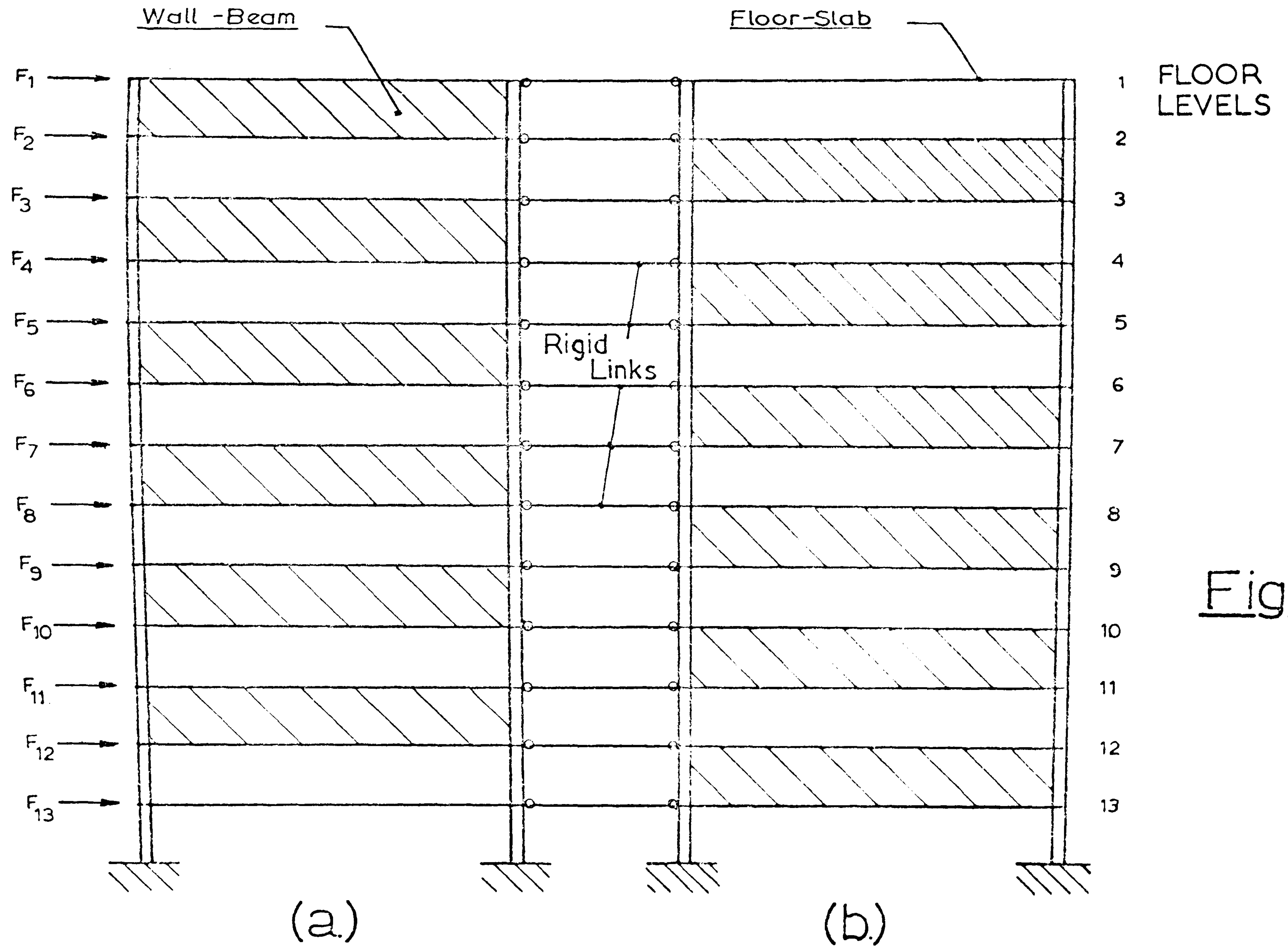

Wall -Beam
Floor-Slab
FLOOR LEVELS
F1
F2
F3
F4
F5
F6
F7
F8
F9
F10
F11
F12
F13
1
2
3
4
5
6
7
8
9
10
11
12
13
Rigid Links
(a.)
(b.)

Fig. C-1

DISTRIBUTION OF LATERAL LOADS

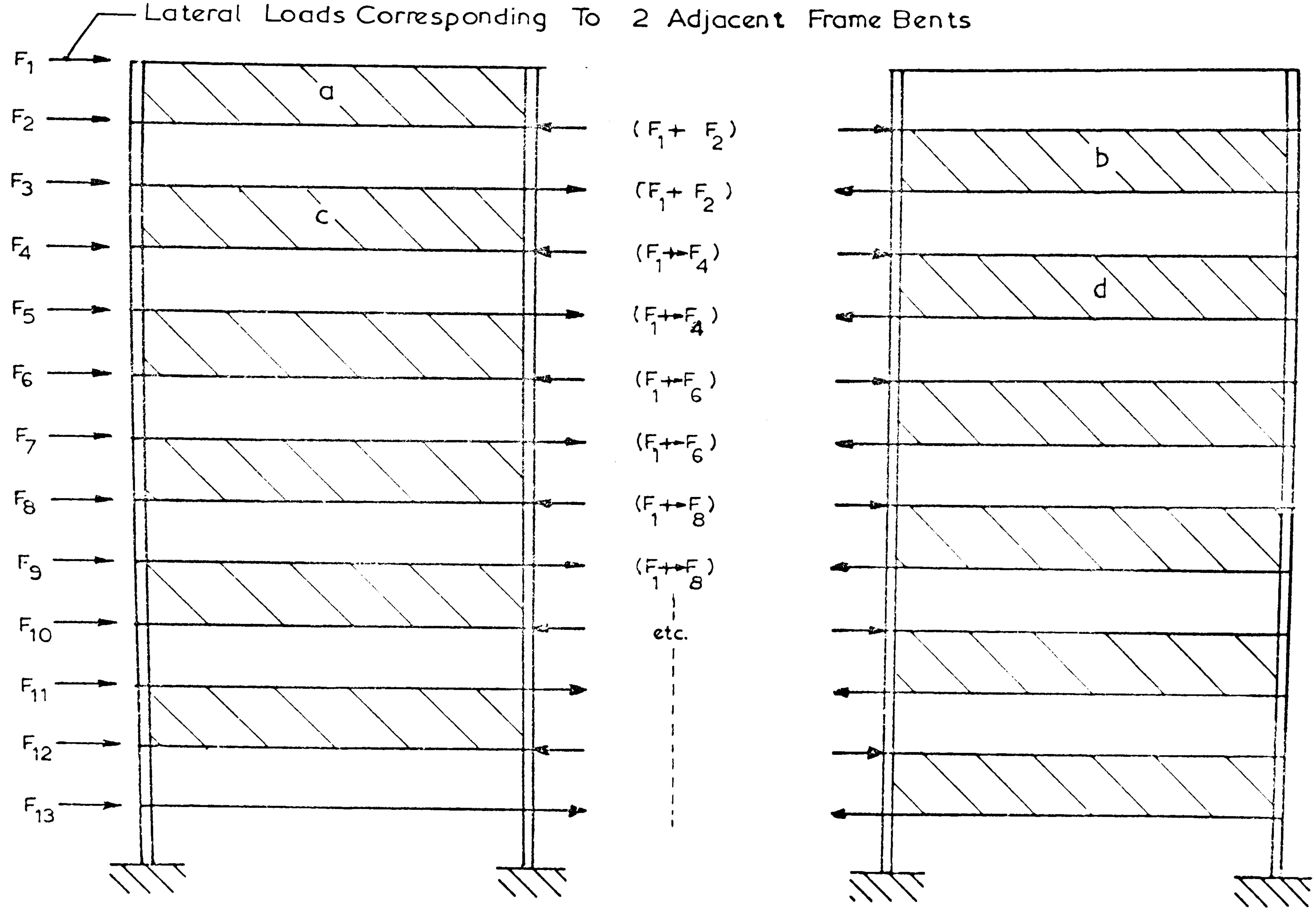


Fig.C -2

NET LATERAL FORCES

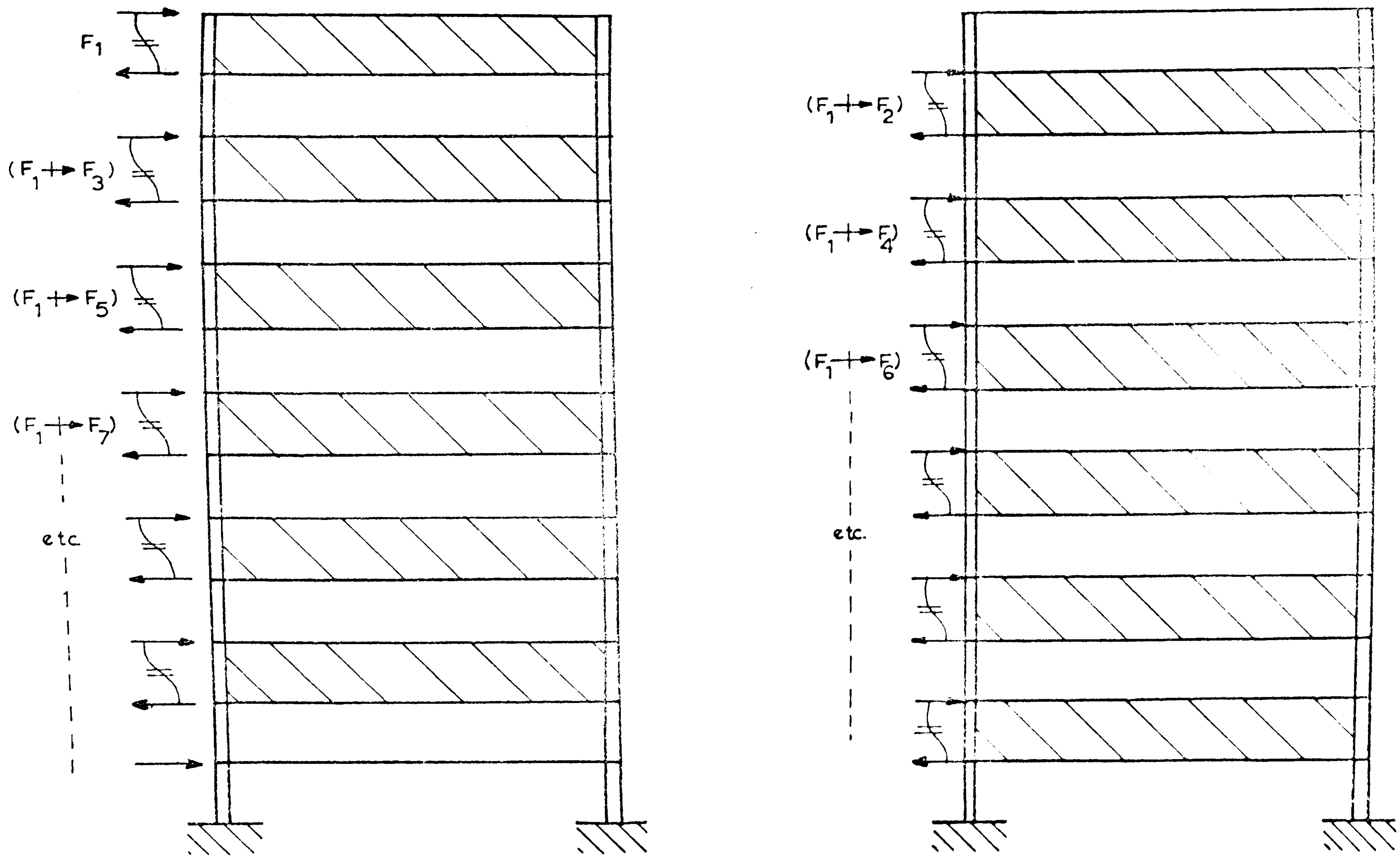


Fig. C -3

APPENDIX D

Estimation of Damping

To use the half-power band-width technique it is necessary to obtain a vibration response wave such as that shown in Fig. D-1 . The half-power points P_1 and P_2 are first evaluated by assigning the peak response to the value Q ($= {}^1/(2\alpha) = (km)^{\frac{1}{2}}/d$). Having a value for Q the value of P_1 and P_2 can now be found from $^Q\sqrt{2}$ and the frequencies corresponding to these points, i.e. W_1 and W_2 can be related to the critical damping using the relationship $(W_2 - W_1) = {}^{W}c/Q$. The percentage critical damping is now found from

$$(W_2 - W_1) = \frac{W_c}{Q} = \frac{W_c}{{}^1/(2\alpha)}$$

$$\therefore \quad \alpha = \frac{(W_2 - W_1)}{2W_c}$$

where k = stiffness $\quad$ W = frequency

d = damping $\quad$ P_i = half-power points

$\alpha = {}^d/d_{crit}$

The pluck test, or free vibration test, on a structure yields a response wave of the type shown in Fig. D-2 . The relationship between the reduction in Amplitude and/

and time for this wave is given by

$$\frac{Y_A}{Y_B} = \frac{Y_B}{Y_C} = \frac{Y_C}{Y_D} = \exp\left(\frac{\pi d}{m w_1}\right) \quad \text{............} \quad (a)$$

$\frac{Y_A}{Y_B}$ is defined as the amplitude reduction ratio where Y_A and Y_B are the amplitudes of any two consecutive maxima on the same side of the time base. The natural logarithm of the amplitude reduction ratio is called the logarithmic decrement.

i.e. $\delta = \ln\left(\frac{Y_A}{Y_B}\right)$

therefore from expression (a)

$$\delta = \frac{\pi d}{m W_1}$$

δ = logarithmic decrement

m = mass

and using the relationship for critical damping and amplitude of free vibration

$$\delta = \frac{2\pi \alpha}{(1 - \alpha^2)^{\frac{1}{2}}}$$

The percentage of critical damping in the fundamental mode of vibration of the structure can therefore be found from the expression

$$\alpha = \frac{\delta}{\left\{4\pi^2 + \delta^2\right\}^{\frac{1}{2}}}$$

Dynamic/

Dynamic Modulus of Elasticity

Provided deep beam effects are eliminated, the first natural frequency of a uniform cantilever can be found from the expression

$$W_1 = \frac{3.516}{L^2} \left(\frac{EI}{\rho A} \right)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(b)}$$

where

W_1 = natural circular frequency

ρ = density of the material

E = dynamic modulus

I = second moment of area

A = cross-sectional area

L = length of beam

Having found the first natural frequency from either resonance testing or pluck tests, the modulus of elasticity in the first mode of vibration can be found from expression (b).

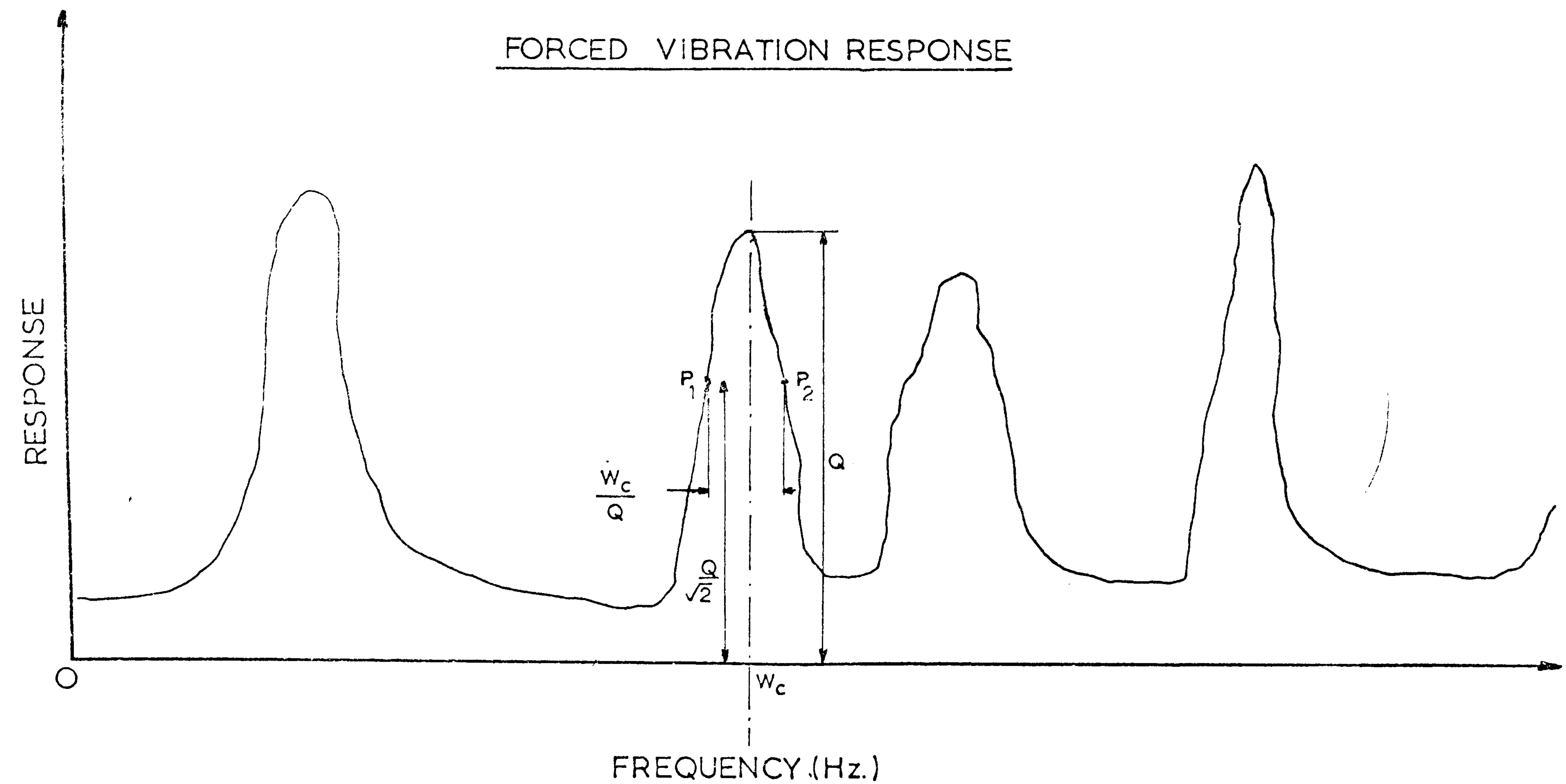

FORCED VIBRATION RESPONSE
RESPONSE
O
FREQUENCY (Hz.)
W_c
$\frac{W_c}{Q}$
P_1
P_2
Q
$\frac{Q}{\sqrt{2}}$

Fig.D-1

FREE-VIBRATION RESPONSE

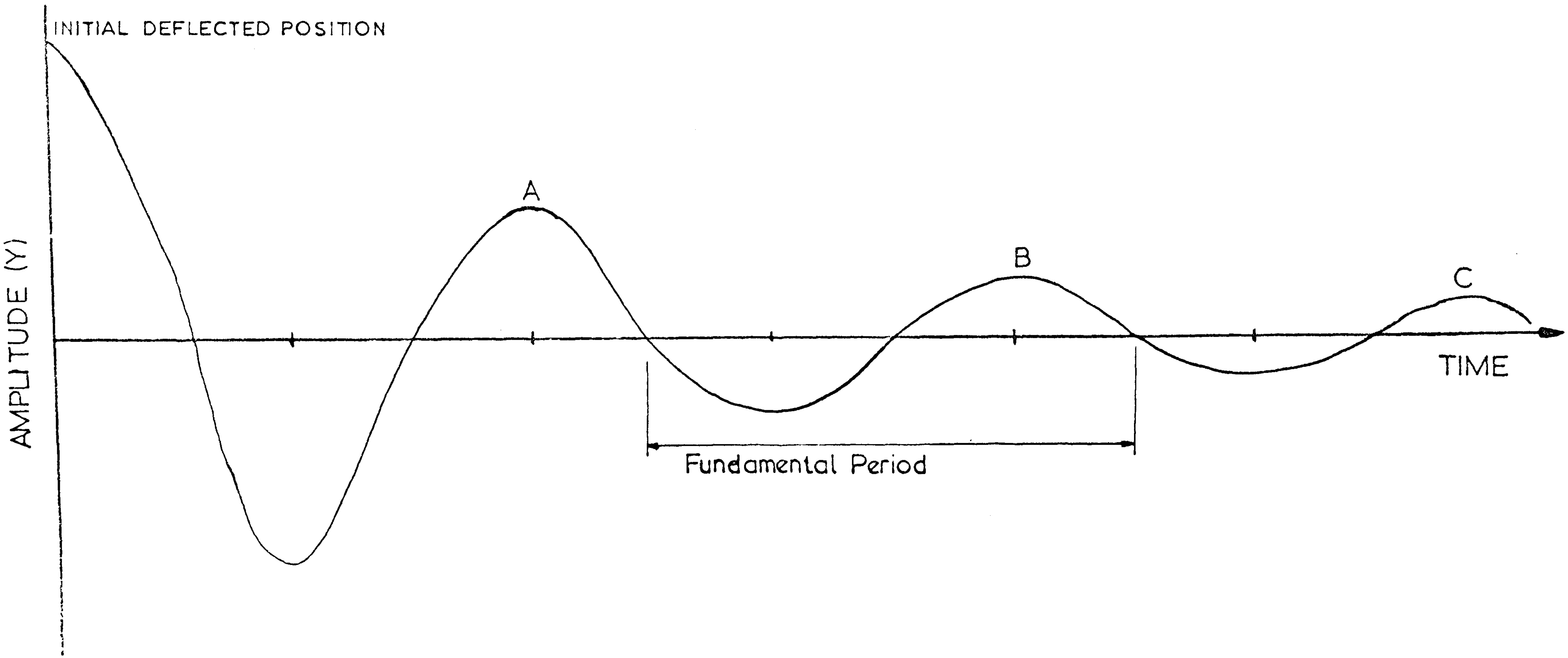
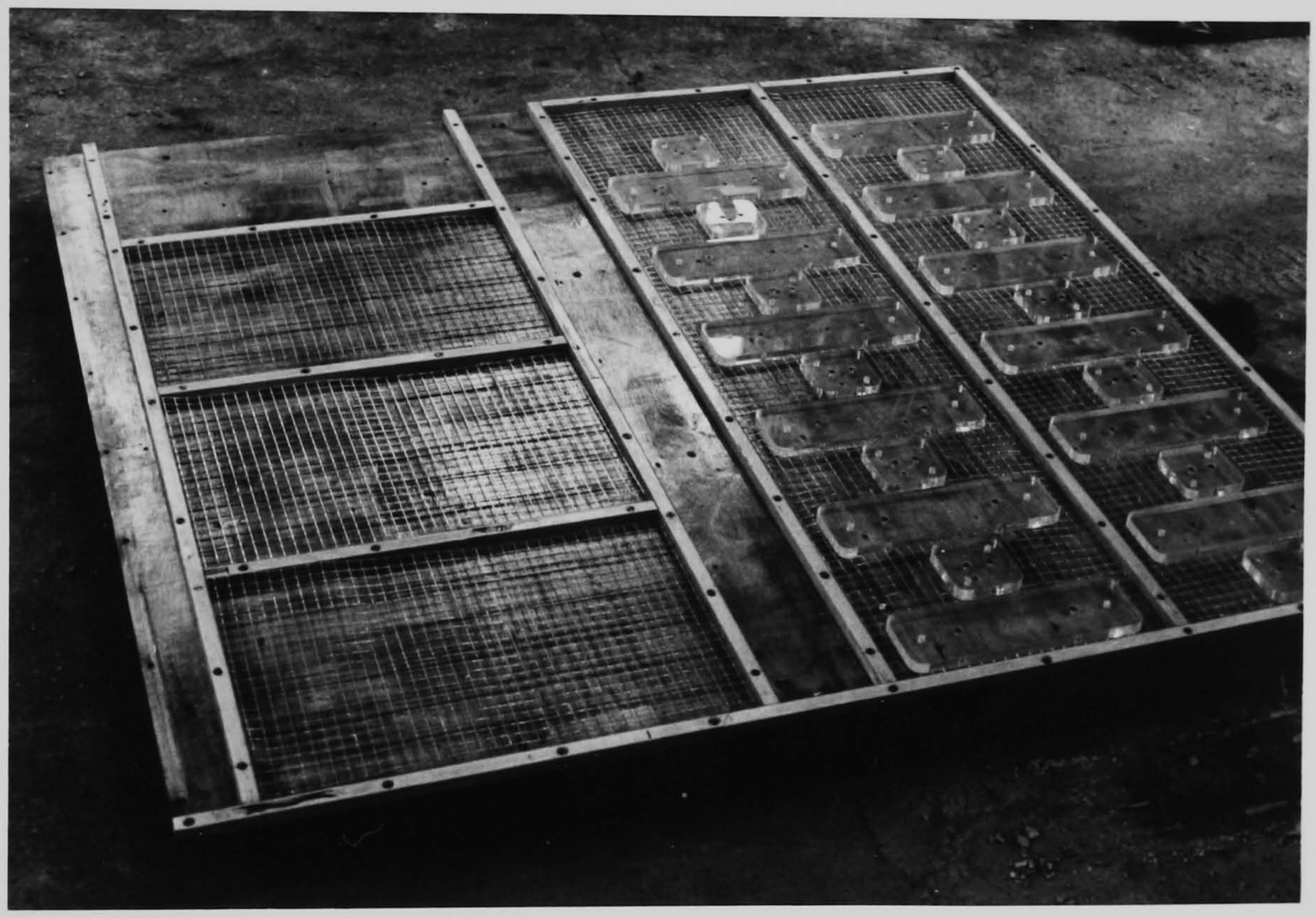


Fig. D-2

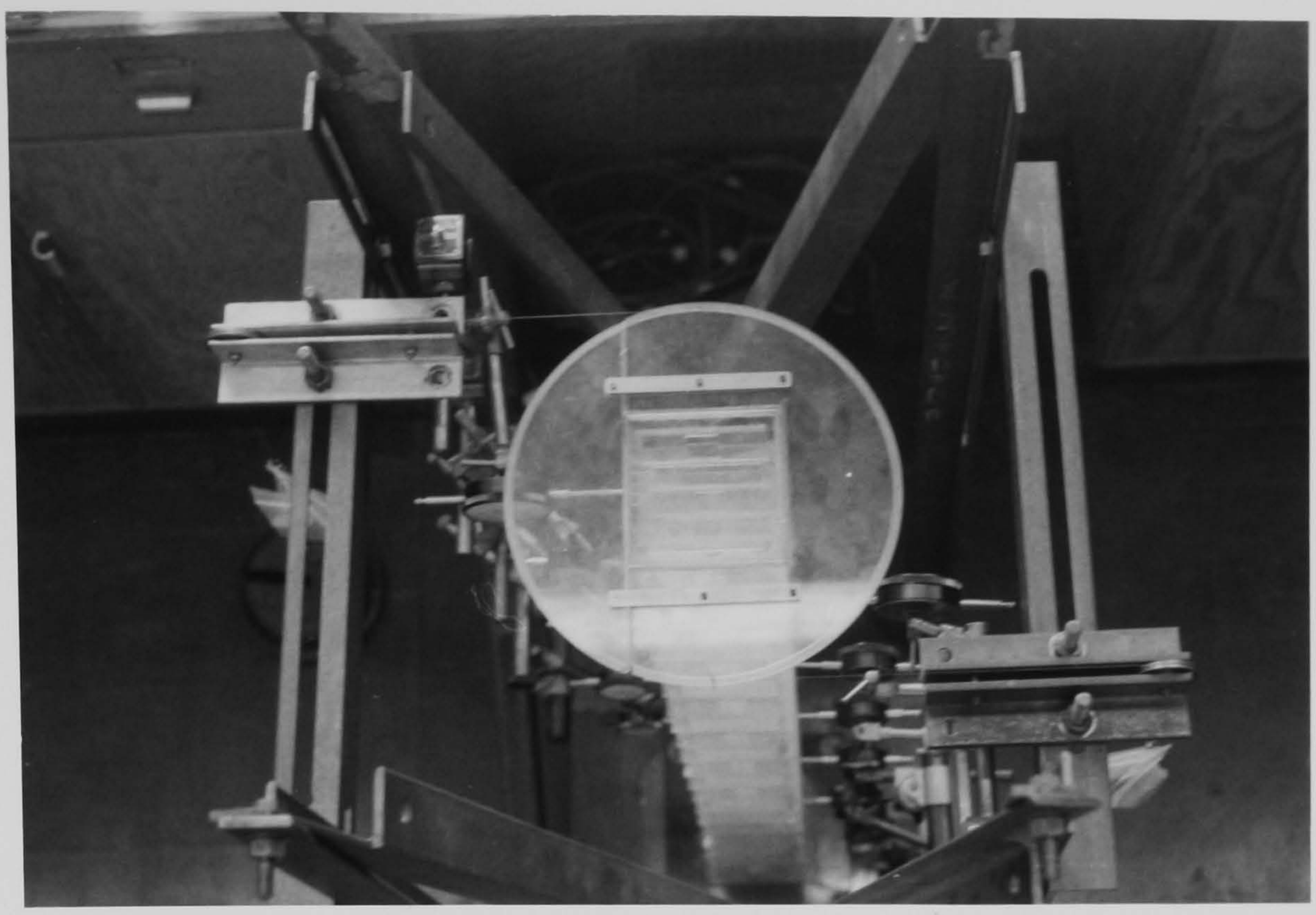
LIST OF PLATES

No. 1



No. 2

No. 3



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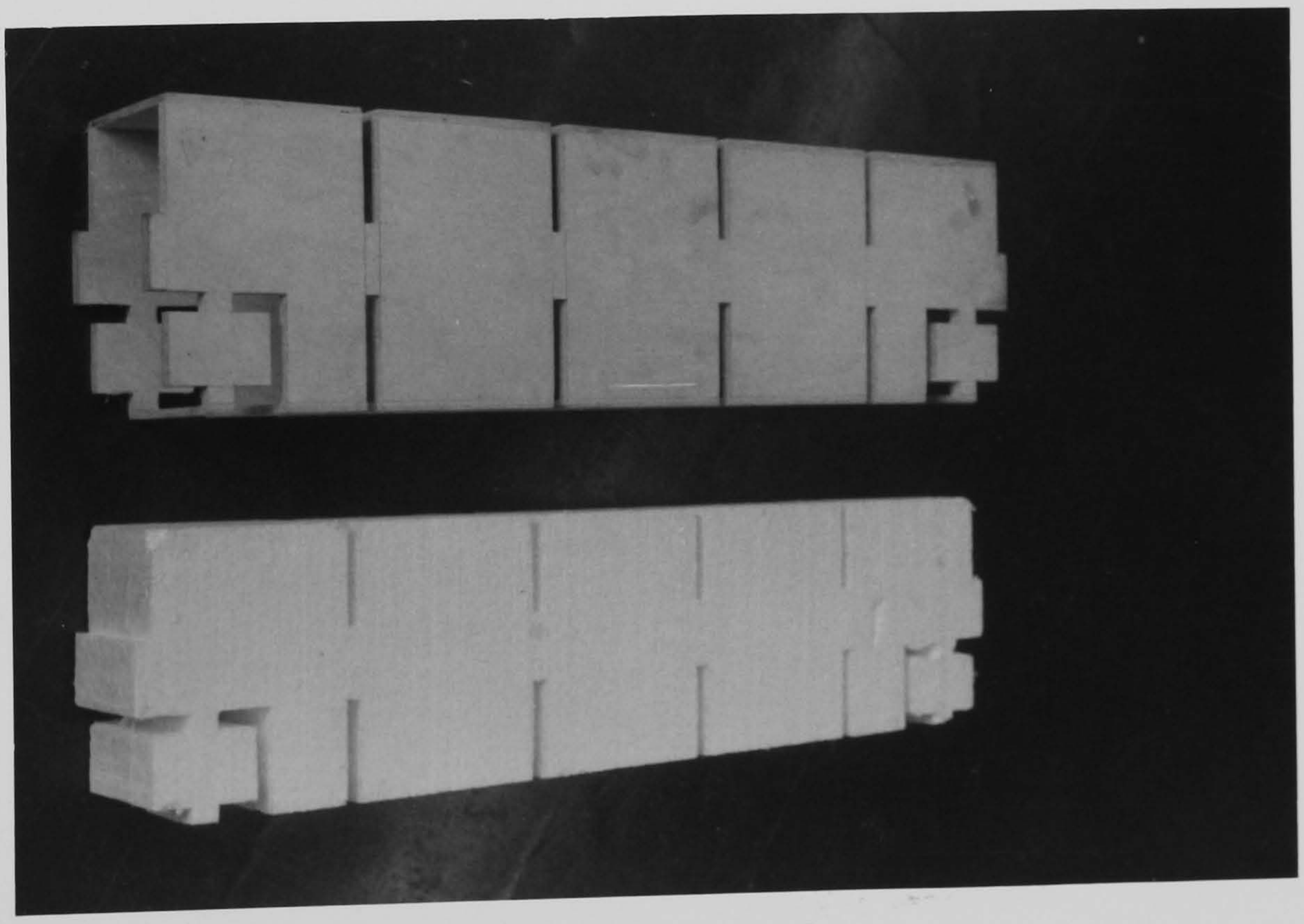
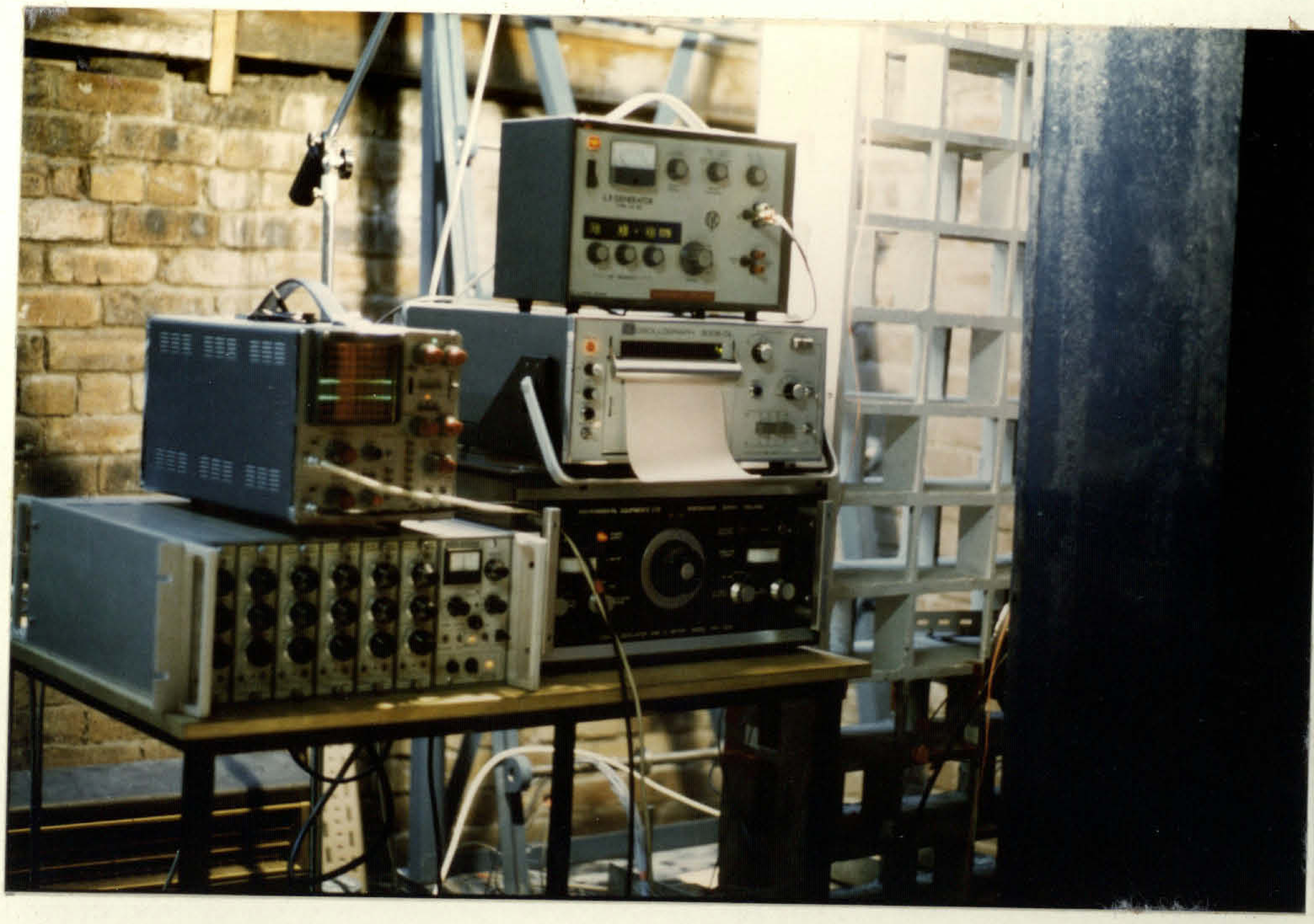


No. 5

No.6

No.7



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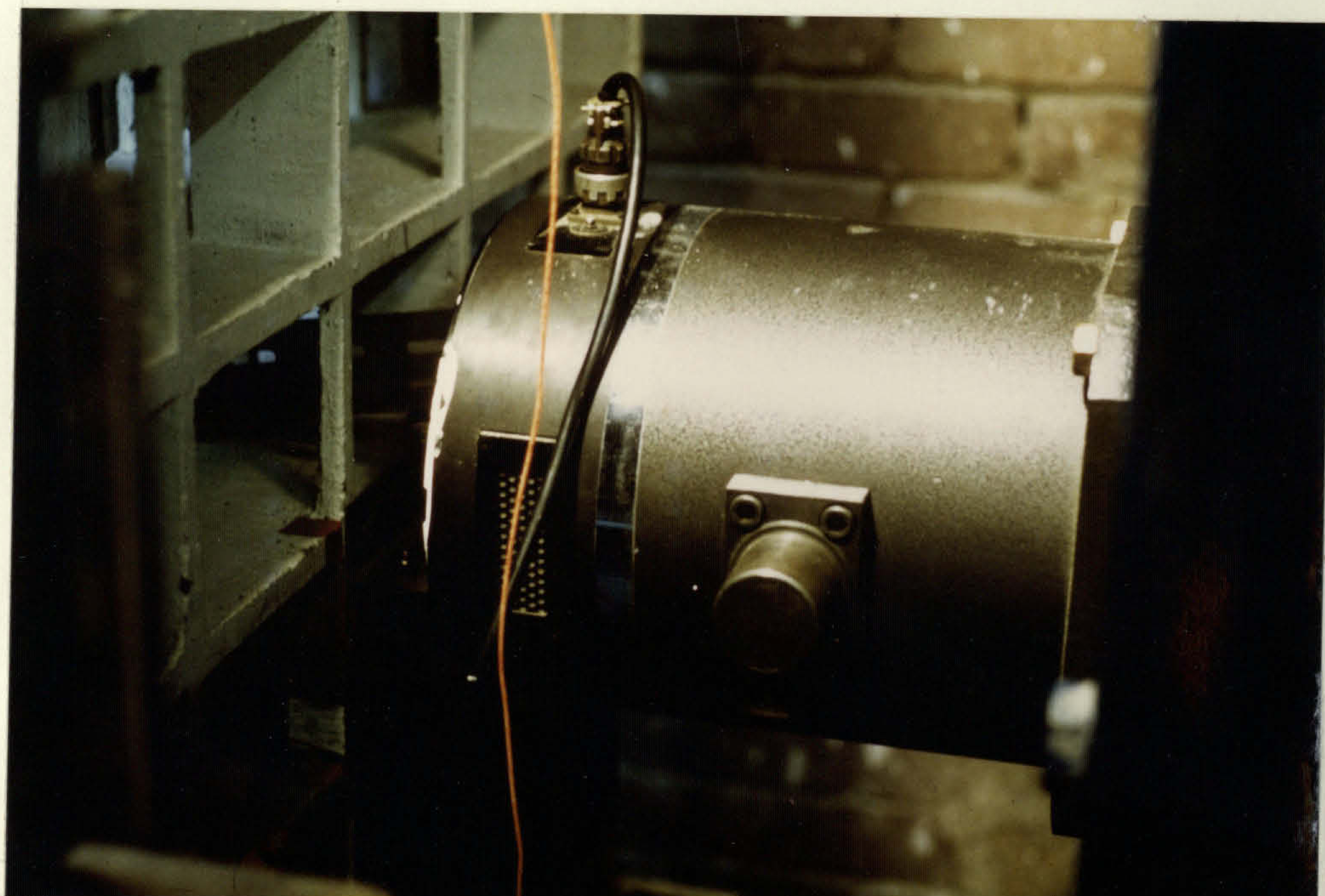

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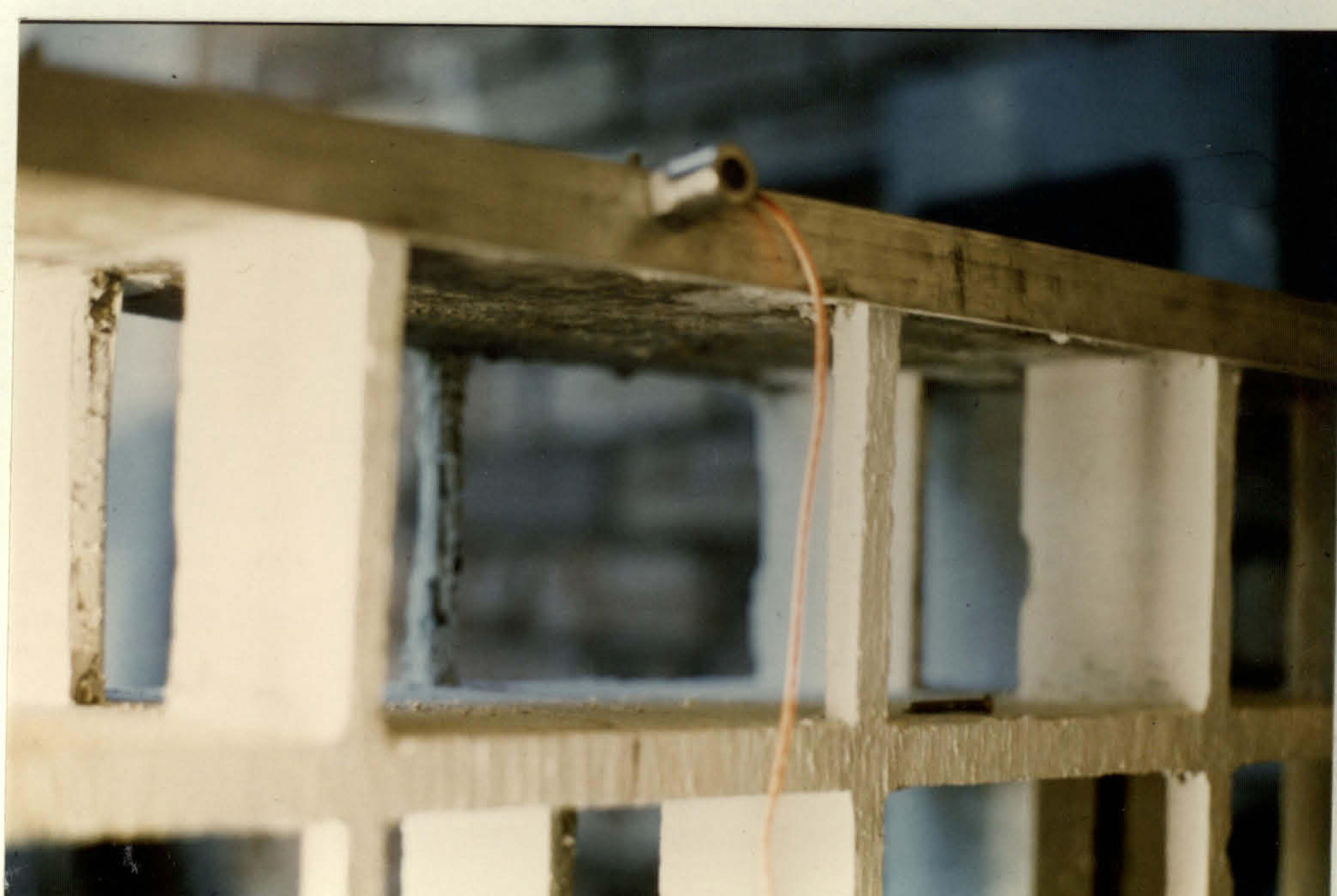

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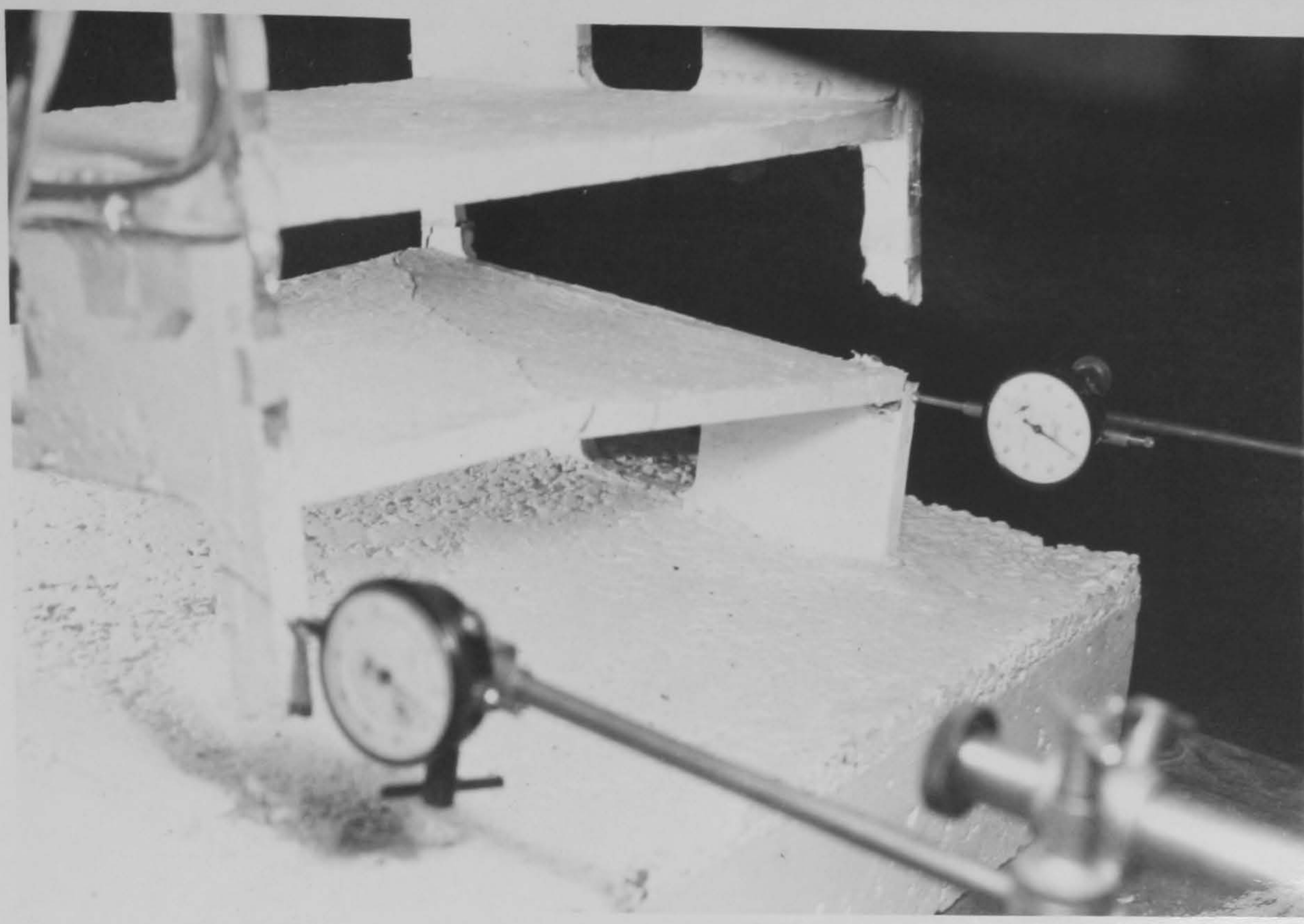

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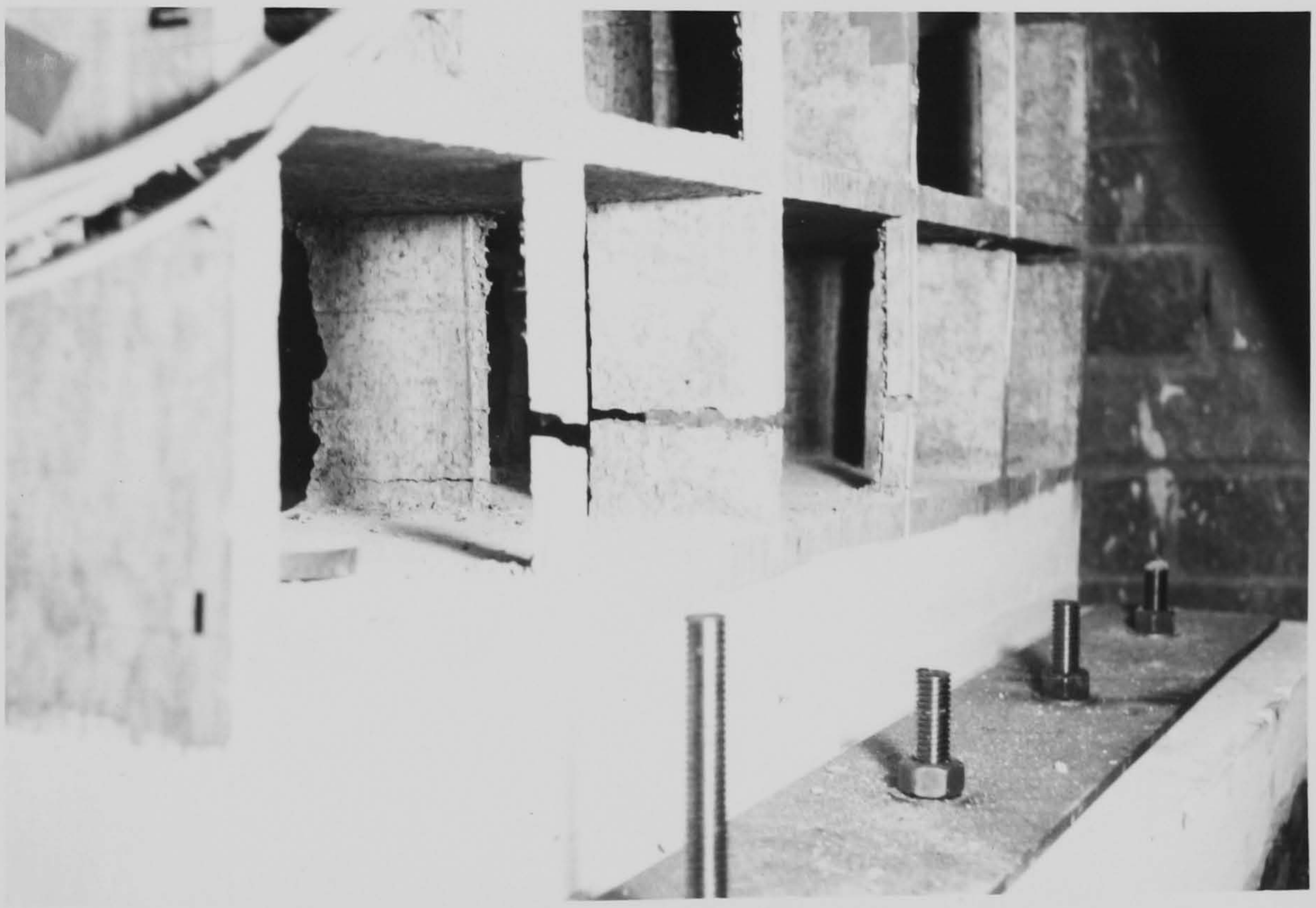

No. 12

No. 13